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THE MISSOURI RIVER BRIDGE at East Omaha, Neb., is to be enlarged and partly rebuilt. The old bridge consists of a double-track draw span of 520 ft., and three combination wood and iron spans of 192 ft. The new work includes a new double-track draw span of 520 ft., and 500 ft. of steel trestle approach. There will also be four new river piers required and several small piers for the approaches. The work will be done partly by the Illinois Central R. R. and the Omaha Bridge & Terminal R. R. Messrs. Waddell & Hedrick, Mems. Am. Soc. C. E., of Kansas City, Mo., have been retained as consulting engineers.

AN OLD WOODEN BRIDGE was successfully moved a distance of about 80 ft. parallel to itself in Munich, Germany, some weeks ago. The bridge, which crosses the river Isar, was built in 1840; it had 11 river spans and two shore spans resting on pile bents. In the course of time the condition of the old bridge had become such that it was resolved to replace it by a stone bridge. The contractors for the new bridge were required to construct a temporary crossing, and they elected to utilize the old bridge for this purpose. Piles were driven in line with the piers and just upstream from them, forming a new set of bents. The old bridge was jacked up from its bents and tracks placed between the main cross timbers of the bridge and the cap timbers of the bents. The track was formed by 5½-in. balls rolling between channel irons. The tackle for moving the bridge consisted of a windlass mounted on the bridge over each pier, with its chain passing out and over a sheave at the upstream end of the new bents, thence back to a fastening on the bridge. The tracks were in place two days under full traffic before the moving was started, and then it was found impossible to budge the structure because the balls had crushed down into the metal of their channel tracks. It was necessary to jack up the bridge again and move the balls out of these depressions; the moving was then proceeded with, and after 14 hours of work the bridge had arrived at its new position. The bridge was actually in motion a time of 1 hr. 40 min.

THE RAILWAYS OF THE WORLD are rated in the "Journal Officiel," of France, for May 25. The figures profess to give only the lines in actual operation in 1900 in the five geographical divisions of the world, and for decades from 1830. Taking the summaries only and translating into miles, the record stands as follows:

	Miles of line open.	Population by millions.	Miles pr 10,000 people.
Europe	175,785	400.0	4.4
America	249,676	141.8	17.6
Africa.	11,450	59.6	1.9
Asia.	25,650	784.6	0.47
Oceanica	15,594	4.73	32.8
Total	478,115	1,390.7	3.5

A CONCESSION FOR A VENEZUELAN RAILWAY, from Maracaibo to Perija, has been recently granted, says U. S. Consul E. H. Plumacher, of Maracaibo. The grant is for 99 years. The length of the road and other physical details are not stated; nor is it mentioned to whom the concession is granted.

THE PILGRIM'S RAILWAY, from Damascus through Medina to Mecca, a distance of 1,600 miles, is halting from lack of funds. The estimated cost is $46,000,000, and the Sultan believed that this sum could be raised entirely among pious Mosselmen. But, up to November last the total subscription amounted to $1,840,000, and only $560,000 of this was paid in cash. Work was commenced last year and about 300 miles are graded after a fashion. The Sultan has now conceived the brilliant business idea of presenting a nickel, silver or gold medal to all subscribers of sums of 1,200, 2,200 or larger amounts in francs. These medals do not attract much attention in Lombard St. and in Wall St., though the assistance of the infidel is asked for. As this railway presents no physical difficulties, other than lack of water, and would be well patronized by pilgrims, it will probably be built, but on a more practical basis of compensation for labor and materials. It is said that the Sultan is quite as much impressed with the military importance of this road as he is with caring for the comfort and health of the pilgrims to Mecca.

THE COMPOUND LOCOMOTIVE'S 25th anniversary is made the subject of a little pamphlet issued by Mr. A. Mallet, of Paris, who was the first inventor of a practicable compound locomotive, as explained in our issues of March 8 and April 19, 1890. His first locomotives were put in service June 2, 1877, concurrently with the opening of the Bayonne & Biarritz Ry. (Eng. News, June 9, 1888). They were built in 1876 at the Creusot works and were of the two-cylinder type, with a special arrangement for admitting live steam into the low-pressure cylinder when starting. These engines have been described in our columns. Mr. Mallet gives the number of compound locomotives in 1902 as nearly 15,000; of these, about 66% are of the type originated in 1876, which he introduced into Russia, Spain, Austria and Switzerland. Out of this total, more than 500 engines belong to the four-cylinder articulated or flexible steam-truck type also designed by Mr. Mallet, one of the first of which was exhibited at the Paris Exhibition of 1889. A large engine of this type was illustrated in our issue of Aug. 15, 1891. During 1877 he also began the construction of compound locomotives provided with an arrangement for varying the admission of steam into the two cylinders separately and according to requirements. This arrangement is in use on a number of two-cylinder and four-cylinder compounds. In 1877 Mr. Mallet was awarded the Fourneyron prize for mechanics by the Institute of France, in recognition of his application of the compound system to locomotives. In the United States there are about 45,000 locomotives, but we have no record of the proportion of compounds. Of the locomotives built within the past year or two, however, about 25 or 30% have been compounds.

A LARGE PROPORTION OF ENGLISH LOCOMOTIVES are 20 years old, and some are 30 to 40 years old, says Mr. Charles Rous-Marten in a paper lately read before the English Society of Engineers. British locomotives only 20 years old, he said, are regarded as comparatively modern, and while the longevity of these engines testified to the use of good material, the fact remained that they were out of date and deficient in the power demanded by modern traffic. The loads of the older engines were limited to five-sixths of that given to modern machines; in other words, six engines and six separate trains had to do the work of five better machines on roads already badly overcrowded; labor cost and working expenses were increased. In comparing American practice, the speaker said that our engines were not expected or desired to last more than 10 or 15 years, and they were replaced by new engines fitted with all modern improvements and possessing a large margin of power. Mr. Rous-Marten questioned the wisdom of using in American locomotives material and workmanship that were inferior—from an English standpoint; but he also thought the extreme longevity of English locomotives was very undesirable, for the reasons given.

THE MOST SERIOUS RAILWAY ACCIDENT of the week was the wreck of a fast passenger train on the New York Central & Hudson River R. R. at Herkimer, N. Y., on July 10. The train ran into the wreckage of a derailed freight train, and was itself wrecked. Very fortunately no one was killed, but several persons were injured.

THE CAMPANILE OF SAN MARCO collapsed on the morning of July 14. This famous old tower stood in the square of St. Mark, Venice, Italy, and for many centuries it had dominated the city, rising high above all other buildings. The tower was about 40 ft. square at the bottom and over 300 ft. high to the figure at the top. It was begun in the year 888, but was completed in its present form only in 1517. The structure was a slightly tapering shaft of brick capped with a tall pyramidal roof, beneath which was the bell loft. The foundation of the tower was of stone, resting on piles capped with several solid courses of timber about 20 ft. below the (present) surface of the ground. The subsoil is clay, with streaks of sand and gravel, to great depth. The foundation is said to have been examined in 1885 and found perfectly sound. The piles were of hemlock, the capping timbers of oak. The cause of the fall is to be investigated. It is the opinion of some that dredging in the canals nearby may have caused a motion of the soil. The New York "Times" quotes Professor Belar, of the Seismic Observatory at Laibach, Austria, as being of the opinion that the tower had been affected by the recent seismic disturbances in northern Italy and that its final downfall was probably caused by the earthquake at Salonika, Turkey. He stated that last Easter he had observed a marked inclination of the Campanile. Part of the brick work of the tower had only recently been repointed, it is said. The tower will doubtless be reconstructed at once. Subscriptions for this purpose in Venice have already reached a large sum, and the Italian Government may aid in the work.

A FATAL TROLLEY CAR COLLISION resulted in the death of one person and the injury of many others on July 9. A crowded excursion car, running special, crashed into an express car running in the opposite direction, on the Hudson Valley Electric Ry. at Lake George, N. Y., with the results above noted. The responsibility for the accident has not been fixed. The motorman of the express car knew nothing of the special until he saw it coming toward him, when he applied the brakes; the other car was going down hill at the time and ran into the express car at full speed.——On July 13 a crowded trolley car of the Union Railway Co. in the borough of the Bronx, New York city, ran away down a grade and jumped the track at a curve below, striking a telegraph pole. A number of the passengers of the car were seriously hurt.——A more serious accident than either of the preceding is reported from Richmond, Va. Two electric cars collided on the Richmond and Petersburg Ry., near Centralia, Va., on July 15, and in the wreck three persons were killed and several others injured.

A FRIGHTFUL MINE DISASTER at Johnstown, Pa., caused the death of over one hundred miners on July 10. At about noon of that day an explosion of gas occurred in the bituminous coal mine of the Cambria Steel Co. Attempts at going to the rescue of those caught in the mine were at once begun, but the great amount of afterdamp much retarded the work. After two days' work about 110 bodies had been recovered, and it is probable that there are only a few, if any, dead left in the mine. State mine inspectors were early on the scene, and they will investigate the accident in order to determine its cause and place the responsibility. Up to the present it has become known that the miners were quite careless in the matter of using naked lights, as also in the matter of blasting. It is said that for some time that section of the workings where the explosion took place had been dangerous, on account of extra large quantities of gas given off.

ANOTHER RECORD-BREAKING RAILWAY RUN is to be chronicled. On July 12, 1902, the new 20-hour train of the New York Central & Hudson River R. R. covered the distance from Brocton, N. Y., to Chicago, Ill., 481 miles, in 460 minutes, which is equivalent to a rate of speed of 63 miles an hour maintained for nearly eight hours. This train, which was put into service only a few weeks ago, is scheduled to make the distance from Chicago to New York in 20 hours, as against 24 hours, the time of the fastest regular trains hitherto. At Buffalo the train was over two hours late, and orders were issued to make up as much of this time as possible, over a clear track. The train arrived at Chicago only about half an hour behind schedule time. This run will probably stand as the record for long-distance high-speed running for some time to come.

REGULATIONS FO[illegible]NG AND CULTIVATING trees in the streets of New York have been adopted by the Park Department. They provide that no tree or ornamental shrub shall be planted in any public street of the city until the tree or shrub and the mold to be placed in the hole for the same has been approved by an employee of the Department. Other rules prohibit cutting or disturbing trees or shrubs, cultivating, fertilizing or paving the earth within 3 ft. of any tree or shrub without a permit granted after inspection by the Department. The rules are declared to have the effect of ordinances, and a fine of not over $50, or in default thereof imprisonment not exceeding 30 days, may be inflicted for violation thereof.

A WATER-WORKS INVESTIGATION at Memphis, Tenn., is to be made by Capt. Arthur Hider, M. Am. Soc. C. E., of Greenville, Miss.; J. A. Omberg, Jr., and A. T. Bell, of Memphis, as a board of engineers, and Haskins & Sells, 30 Broad St., New York city, as expert accountants. The efficiency of the present plant, owned by the Artesian Water Co., will be tested, estimates of the value of the company's works and of the cost of new works will be made, and the water rates of Memphis and other cities will be compared.

WATER PURIFICATION AT PITTSBURG, PA., was indefinitely postponed by the recent action of the common council of that city. The reason given for such a step is that the estimated cost of the works exceeds the bond issue authorized for that purpose. The real reason, it is said, is that the new reform council is unwilling to authorize work laid out by the opposing political party.

A COMBINATION TRANSFER TABLE AND CAR ELEVATOR.

An entirely novel style of car transfer table has recently been constructed, which not only carries cars from track to track upon the surface, but also elevates the car to the tracks upon the second story of a building. The machine is in use at a car building plant, but the arrangement suggests a means of economizing space at railway car shops, car storage sheds, freight terminals, etc. It consists essentially of a steel frame mounted on wheels, and having a series of built-up steel columns surmounted by rope sheaves. The platform or deck is suspended by wire cables, which pass over the upper sheaves and down over lower sheaves to winding drums operated by an electric motor.

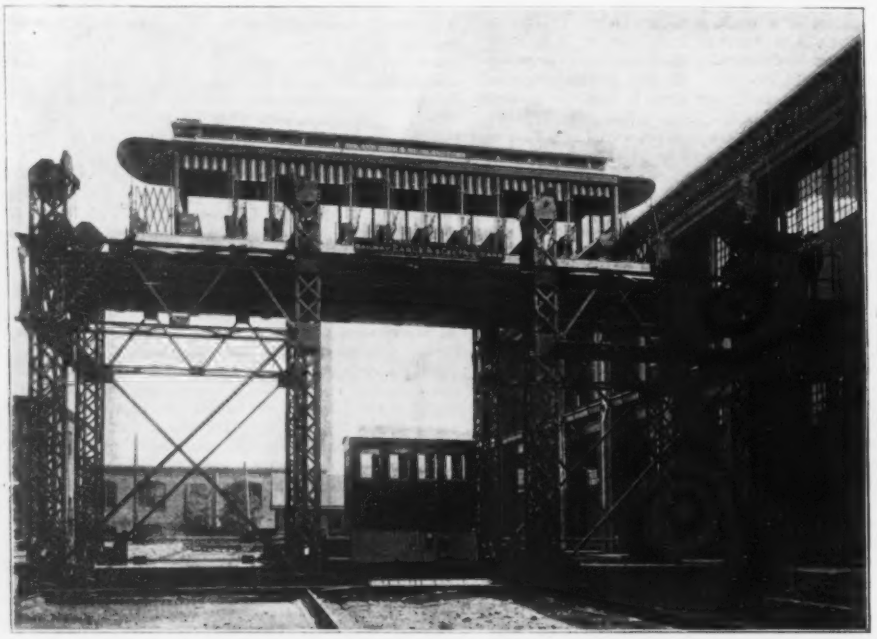

Fig. 1. Side View.

Fig. 2. End View.

CAR TRANSFER TABLE AND ELEVATOR FOR THE J. G. BRILL CO.; PHILADELPHIA, PA.
Geo. P. Nichols & Brother, Chicago, Ill., Designers and Builders.

The transfer table in question was built for the J. G. Brill Co., of Philadelphia, Pa., builders of street railway cars, and it was specially designed to serve a two-story paint shop built to economize the limited ground space available. The finished cars are hauled from the car shop on a track leading to the transfer table, upon which the car is run. The table is then moved lengthwise of the building (which is about 200 ft. long) to the shop track on which the car is to be set. If the track is on the first floor, the car is run directly upon it from the table in the ordinary way. If the track is on the second floor, the winding drums are put in operation and the deck and car are raised, the difference in elevation being 19 ft. 6 ins. Fig. 1 is a side view, showing the deck raised. Fig. 2 is an end view, with the deck lowered, and showing the construction of the columns and the cross braces.

The total length of the table is 50 ft., with an 8-ft. overhanging platform at the upper level to cover the space between the shop floor and the deck of the table. It runs on eight wheels on four lines of rails laid upon timber stringers. The hoisting capacity is for a car weighing 30,000 lbs., but the test load was approximately 40,000 lbs. The hoisting is effected by eight three-part cables, and the hoisting speed is 15 ft. per minute. The table is provided with a winding drum for hauling cars on and off when the table is at either its upper or lower position. In the operator's cab are placed the controller, brake lever and electrical appliances.

The motive power is a single electric motor, the hoisting mechanism, the table travel and winding drum motions being obtained through three friction clutches operated by levers in the cab, these levers being interlocked so that only one clutch can be engaged at a time. Automatic stops are provided at the top and bottom so that the current is cut off and the brake set at the proper point in either case. The mechanism between the motor and the hoisting drums is a train of spur gearing. A mechanical load brake is provided in this train, so that when the load is being lowered this brake is engaged and a slight amount of power is required from the motor to overcome the friction of the brake. This device makes it impossible for the load to get beyond the control of the operator. By means of the solenoid motor brake the momentum of the armature is checked as soon as the current is cut off by the controller of the automatic stop. An auxiliary hand brake is also provided as an additional precaution. Power is taken from a trolley wire laid along the front of the building, just below the level of the upper floor.

This table was designed, built and erected by Geo. P. Nichols & Bro., Monadnock Block, Chicago, to whom we are indebted for photographs and other information.

RAILWAY GRADE CROSSING PROTECTION IN TEXAS.

The State Legislature of Texas passed a law last year providing for the protection of railway grade crossings. This law requires the railways to put in interlocking plants, but allows them to run their trains without stopping at crossings thus protected. An important provision is to the effect that in cases where a new crossing becomes necessary it shall be the duty of the Railroad Commission to ascertain what manner of crossing will occasion the least probable injury to the first or existing road. If it appears that it is reasonably practicable to avoid a grade crossing, the Commission may prohibit the construction of such a crossing. If the crossing is permitted, the railway desiring it will be compelled to interlock or protect it by safety devices as designated by the Railway Commission, and to pay all the costs of the plant and its installation. The act does not apply to sidetracks, but imposes a penalty of $500 per week in case of refusal or neglect to comply with the orders of the Commission.

This act took effect on July 9, 1901, and during the early part of the present year conferences have been held between the Railroad Commission and the representatives of the railway companies. The general opinion expressed by the latter was that conditions in Texas are not yet such as to demand the extensive installation of interlocking devices, and that to compel their installation at once at all crossings would be imposing an unreasonably great burden upon the railways. The Commission concurred somewhat in this view, but ordered that 27 of the most important crossings should be equipped with interlocking plants within one year. This is regarded as something of an experiment, and their success will largely determine the future action of the Commission. At the present time there are only three interlocking plants in operation in the State, all of which have been recently installed.

This order was made on June 5, 1902, and requires that the interlocking plants shall be completed and put in operation (under the permit of the Commission) by June 30, 1903. It provides that the cost of the plant and installation shall be borne equally by the railway companies involved in each case, while the expense of operation and maintenance is to be shared in proportion to the number of levers necessary to operate the devices for each road. The order does not, however, set aside any existing agreement between railways as to the division of expense in such matters.

Under the terms of the order, all railway grade crossings not now protected by interlocking devices shall be equipped with crossing gates of such design and so arranged that whenever they are broken down by engines or cars it can at once be determined which railway is responsible. These gates are to be operated under strict rules by each railway company, and are to remain in use until the crossings are equipped with interlocking plants. The cost of construction and operation will be borne equally by the railway companies involved at each crossing. All trains are required to stop before proceeding over these crossings. One clause of the order provides that whenever it has been determined to be practicable to separate the grades at any crossing ordered to be interlocked, and the railways desire to separate the grades, the order will be suspended as to the interlocking, but the separation of grades must be effected within the time specified by the order. The railway companies must report on April 30, 1903, the average number of trains operated daily over the crossings not interlocked or ordered to be interlocked, and also the location of crossings where it has been decided that separation of grades is practicable and shall be made before June 30, 1904.

Mr. John H. Reagan is Chairman of the Commission, Mr. E. R. McLean Secretary, and Mr. R. A. Thompson, Austin, Tex., is Chief Engineer. To the latter we are indebted for much of the above information.

The rules and regulations of the Texas Railroad Commission, governing the installation, maintenance and operation of interlocking devices, are

based largely upon those of the Illinois Railroad Commission, and are given below, somewhat condensed. Special attention may be called to the color system specified. The rules require the filing of plans of each crossing, with profiles of the main lines for one mile on each side. When the interlocking plant has been inspected and approved by the Commission, a permit will be issued authorizing trains to run over the crossing without stopping, but this permit may be canceled if the plant is not properly maintained and operated. Monthly reports are to be made as to the condition of the interlocking plants, and special reports of any accidents at interlocking crossings.

REQUIREMENTS AND RECOMMENDATIONS FOR INTERLOCKING SYSTEMS: TEXAS RAILROAD COMMISSION.

STYLE OF SIGNALS.—It is ordered that a uniform system of signals by means of semaphores and lights be adopted by all railroad companies in order that there shall be no confusion at any time with regard to the meaning of such signals. It is further ordered that the red light be used to indicate "Danger, Stop," the green light to indicate "Safety, Proceed," and the yellow light to indicate "Caution."

All signals must be of the semaphore type, and so constructed as to go to the danger position by force of gravity in case the connections between the operating lever and signal are broken. All signals must be provided with a lamp showing front lens properly focused and a back light, except as otherwise provided.

HOME SIGNALS.—The home signal must, when practicable, be located on the engineman's side of the track it governs, and must be not less than 50 ft. nor more than 200 ft. in advance of the point it governs, except when special conditions exist. The signal must point to the right of the track it governs and must have a square end.

When the derail or facing point or crossing is set against the train movements governed by the home signal, the signal must be locked in a horizontal position, showing red by night. When the track it governs is clear and safe for the passage of trains, the signal must be inclined at an angle of about 60° from the horizontal, showing green by night.

In case two signal arms are used on the home signal post, the top arm must in all cases govern the main or high speed route and the lower signal the diverging route or routes. In mechanical interlocking systems the home signal must be worked by pipe connections.

DISTANT SIGNALS.—The distant signal must be located not less than 1,200 ft. in advance of the home signal with which it operates, except under such special cases as may be approved by the Commission. It must be placed on the same side of the track with its arm pointing in the same direction as the home signal. It must be distinguished by a notch cut in the end of a semaphore arm. It must be so arranged and connected with the home signal that it will be held in a horizontal position, showing yellow, or "Caution," color by night to approaching trains when the home signal indicates danger. The distant signal must be worked by two lines of wire.

SWITCH INDICATORS.—Rotating indicators, known as pot or disk signals, must only be used as switch indicators, operated with the switch.

DWARF SIGNALS.—Dwarf signals having a small area and suitably adapted as to height must be similar in design and location to the home signal. They must be used to govern movements on secondary tracks or movements against the current of traffic on main tracks where reverse movements become necessary and where necessary in yards.

BRACKET POSTS.—Bracket posts must be used in all cases where it is necessary to signal trains on different tracks, operated in the same direction, from the same post; the position of the posts on the bracket to correspond to the position of the track on which movements are to be governed.

GENERAL ARRANGEMENT OF SIGNALS.—The operator in the tower must be able to see the arms and the back lights of all signals; the back lights on the lamps to be made as small as practicable, having regard to efficiency. When the front lights are visible to the operator in the tower, no back lights will be required. If from any unavoidable cause the arm or light of any signal cannot be seen by the operator, a repeater or indicator must be provided in the signal tower.

DERAILS.—The derailing point shall be located on high speed tracks, not less than 400 ft. in advance of the crossing or fouling point which it is intended to protect where the grade is practically level; but in case of a descending grade towards the crossing or fouling point, the derailing point must be located at such a distance from the crossing or fouling point as to give the same measure of protection that is required for level approach. Guard rails will be required at all derails, to begin before the derail is reached, and to be of such length as the Commission may approve.

On all secondary tracks the derail point shall be located so as to give the same measure of safety required for high grade tracks, and in no case less than 150 ft. from the crossing or fouling point.

In case a spur or switch track connects with the main track between the derail and the crossing which it protects, the spur or siding shall be provided with a derail.

In case of double track crossings where trains are moved on each track, as a rule, in one direction, a derail shall be provided for back-up movements, and for the further purpose of insuring clearance of crossing before clearance signal can be given on opposing route. The back-up derail must be placed not less than 150 ft. nor more than 300 ft. from such crossing.

In mechanical interlocking plants all derails and point switches, whether facing or trailing, must be worked either by iron or steel pipe not less than 1 in. inside diameter, of standard specifications.

LOCKS.—All slip switches, movable point frogs and derails must be locked either by a separate line of connections from those used to move such slip switches, movable point frogs or derails, or by double pointed switch and lock movement of approved pattern. Where the double pointed switch and lock movement is used on high speed tracks, it must be in connection with a bolt lock operated with the home and distant signals which indicate the position of the facing point.

DETECTOR BARS.—All derails, facing point switches and other fixtures changing the route or impeding the progress of trains shall be protected by detector bars each at least 50 ft. in length. The first interval of the movement of the switch lever which withdraws the locking pin must, at the same time, raise the detector bar above the level of the rail. The final movement of the switch lever must advance the detector bar to its normal position, the level of the rail. If the detector bar is not worked on the switch lever it must be actuated before the switch is moved in either direction.

When it is practicable, detector bars or electric locking will be required at each crossing. Crossing bars must be interlocked with the movement that operates the derails to insure a clear crossing before an opposing route can be set or signal given.

ARRANGEMENT OF LEVERS.—In all mechanical interlocking devices, the levers by which points and signals are worked must be grouped in a tower, and so far as may be practicable the levers must be so arranged that those used in any route combination shall be near together, preference being given to combinations most often set up. The levers must be numbered from left to right. The visible parts of the levers above the machine, except the finished part of the handle, should be painted as follows: Switch levers, black; lock levers, blue; switch and lock levers, black and blue; home signal levers, red; distant signal levers, green, and movable point frog levers, yellow.

PRELIMINARY LOCKING.—The locking must be actuated by the action of the latch rod or by a device performing a similar service in advance of the first movement of the lever. The first act in reversing a lever must lock the levers of all conflicting routes.

LOCKING OF LEVERS.—The levers must be so arranged that while the signals are in their normal position (at danger), the levers operating points shall be free to move. Provided, however, that the preliminary act of reversing any lever shall lock all signal levers controlling opposing routes. The arrangement of locking must be such as to make it impossible for the signalman to lower the signal for the approach of the train until he has first set the points in the proper position for it to pass over the route governed by such signal. The locking must be so devised as to make it impossible for the signalman to exhibit at the same movement any two signals or combination of signals that can lead to a collision.

AUTOMATIC COMPENSATION.—Each line of pipe must be automatically compensated. Such automatic compensators must be located at such intervals in the line as to completely provide for expansion and contraction at various temperatures.

RECENT PROGRESS IN AMERICAN BRIDGE CONSTRUCTION.*

By Henry S. Jacoby.

The application of scientific principles to the construction of bridges is more complete to-day than ever before. This statement applies to the specified requirements which the finished structure must fulfil, the design of every detail to carry the stresses due to the various loads imposed, the manufacture of the material composing the bridge, the construction of every member in it, and finally the erection of the bridge in the place where it is to do its duty as an instrument of transportation.

The form of loading for bridges almost universally specified by the railroads of this country consists of two consolidation locomotives followed by a uniform train load. These loads are frequently chosen somewhat larger than those that are likely to be actually used for some years in advance, but sometimes the heaviest type of locomotives in use is adopted as the standard loading. The extent to which the specified loadings have changed in eight years may be seen from the following statement based on statistics compiled by Ward Baldwin and published in the "Railroad Gazette" for May 2, 1902.

Of the railroads whose lengths exceed 100 miles, located in the United States, Canada and Mexico, only 2 out of 77 specified uniform train loads exceeding 4,000 lbs. per lin. ft. of track in 1893, while in 1901, only 13 out of 103 railroads specified similar loads less than 4,000 lbs. In 1893, 37 railroads specified loads of 3,000 lbs. and 20 of 4,000 lbs., while in 1901, 4,000 lbs. was specified by 50, 4,500 lbs. by 14, and 5,000 lbs. by 17 railroads. The maximum uniform load rose from 4,200 in 1893 to 6,600 lbs. in 1901.

In a similar manner in 1893 only 1 railroad in 75 specified a load on each driving wheel axle exceeding 40,000 lbs., while in 1901 only 13 railroads out of 92 specified less than this load. In 1893 only 21 of the 77 railroads specified similar loads exceeding 30,000 lbs. The maximum load on each driving wheel axle rose from 44,000 lbs. in 1893 to 60,000 lbs. in 1901.

Experience having shown the great advantage of more uniformity in various details and standards relating to the manufacture of bridges both in reducing the cost and the time required for the shop work, an effort was begun to secure more uniformity in the requirements for the production and tests of steel, which is the metal now exclusively employed in bridges.

The thorough digest of these results of scientific research and practical tests, and the preparation and adoption of standard specifications for different classes of material, are confidently expected to eliminate many old requirements which are proved inefficient in securing the results for which they were originally intended, and to incorporate in the specifications only the essential requirements by which the character of the product may be determined with sufficient precision for its actual duty.

With greater uniformity in the physical, chemical and other requirements for steel, as determined by standard tests, the unit stresses to be prescribed for the design of bridges will naturally approach to a corresponding uniformity. To what extent this is desirable may be inferred from the fact that the application of several of the leading specifications to the design of a railroad bridge under a given live load yields results which may vary by an amount ranging from zero to 25% of the total weight.

In the revision of specifications a decided tendency is observed to simplify the design by making an allowance for impact, vibration, etc., by adding certain percentages to the live load according to some well-defined system. It needs but relatively little experience in making comparative designs of bridges under the same loading, to show the advantage of this method over that in which the allowance is made in the unit stresses according to any of the systems usually adopted in such a case. Not only are the necessary computations greatly simplified, but the same degree of security is obtained in every detail of the connections as in the principal members which compose the structure.

Experiments on a large scale are very much needed to determine the proper percentage of the live load to be allowed for the effect of impact, so as to secure the necessary strength with the least sacrifice of true economy.

An investigation might also be advantageously made to determine the proper ratio of the thickness of cover plates in chord members which are subject to compression, to the transverse distance between the connecting lines of rivets. The same need exists in regard to the stiffening of the webs of plate girders, concerning which there is a wide variation in the requirements of different specifications.

A movement which has done much good during the past decade and promises more for the future is that of the organization of bridge departments by the railroad companies. The great economy of making one design rather than to ask a number of bridge companies to make an equal number of designs, of which all but one are wasted, is the first advantage; but another of even greater significance in the development of bridge construction is that which arises from the designs being made by those who observe the bridges in the conditions of service and who will naturally devote closer study to every detail than is possible under the former usual conditions. The larger number of responsible designers also leads to the introduction of more new details to be submitted to the test of service, which will indicate those worthy of adoption in later designs. For the most important structures consulting bridge engineers are more frequently employed than formerly, when so much dependence was placed upon competitive designs made by the bridge companies.

A dozen years ago but few plate girders were built whose span exceeded 100 ft., the maximum span being but a few feet longer than this. To-day such large girders are very frequently constructed and the maximum span has been increased to 126 ft. between centers of bearings. This is the span of the large plate girders of the viaduct on the Riverside Drive in New York city, erected in 1900. The longest railroad plate girder was erected about the same time on the Bradford Division of the Erie R. R., its span being 125 ft. 2½ ins. The heaviest plate girder is the middle one of a four-track bridge on the New York Central R. R. erected last year

*Extracts from the address of the Vice-President and Chairman of Section D—Mechanical Science and Engineering—of the American Association for the Advancement of Science at the Pittsburg Meeting, June 28—July 3, 1902.

near Lyons, N. Y. Its weight is 103 tons, its span 107 ft. 8 ins., and its depth out to out 12 ft. 2 ins.

Although solid metal floors built up of special shapes were first introduced into this country 15 years ago, their general adoption has taken place largely within the past decade on account of their special adaptation to the requirements of the elevation of tracks in cities. Solid floors may not only be made much shallower than the ordinary open type, thereby reducing the total cost of the track elevation, but they also permit the ordinary track construction with cross-ties in ballast to be extended across the bridge, thus avoiding the jar which otherwise results as the train enters and leaves the bridge, unless the track is maintained with extraordinary care.

The necessity for bridges of greater stiffness under the increased live loads has also led to the use of riveted bridges for considerably longer spans than were in use six or seven years ago. The use of pin-connected trusses for spans less than about 150 ft. is undesirable for railroad bridges, on account of the excessive vibration due to the large ratio of the moving load to the dead load, or weight of the bridge itself.

While riveted bridges are now quite generally used for spans from 100 to 150 ft., they have been employed to some extent up to 181½ ft. The recent forms of riveted trusses do not, however, conform to the general character of European designs, but embody the distinctively American feature of concentrating the material into fewer members of substantial construction. With but rare exceptions the trusses are of the Warren, Pratt and Baltimore types with single systems of webbing. At a distance where the riveted connections cannot be distinguished, the larger trusses have the same general appearance as the corresponding pin bridges.

The recent examples of viaduct construction with their stiff bracing of built-up members and riveted connections exhibit a marked contrast to the older and lighter structures with their adjustable bracing composed of slender rods.

While the elevated railroads which have been built recently, also embody many of the characteristics of the best viaduct construction, special study has been given to improve their esthetic effect. The use of curved brackets, of connecting plates whose edges are trimmed into curves so as to reduce the number of sharp angles, and of rounded corners of posts, constitute some of the means employed. The results are seen in the structures of the Boston Elevated R. R. and in some of the latest construction in Chicago.

The longest span of any simple truss in America is that of the bridge over the Ohio River at Louisville, erected in 1893. Its span center to center of end pins is 546½ ft. Since that time several other bridges of this kind have been built, which are considerably heavier, although their spans are somewhat shorter. The most noteworthy of these are the Delaware River bridge on the Pennsylvania R. R., near Philadelphia, and the Monongahela River bridge of the Union R. R. at Rankin, Pa., both of which are double-track bridges. The Delaware River bridge was erected in 1896, each one of its fixed spans having a length of 533 ft. and containing 2,094 tons of steel. The Rankin bridge was erected in 1900. Its longer span has a length of 495 ft. 8⅛ ins. between centers of end pins and contains about 2,800 tons of steel, making it the heaviest single span ever erected. It may also be added that the locomotive and train load for which this bridge was designed is the heaviest that has yet been specified.

The recent changes in the details of pin-connected truss bridges have been mainly the result of efforts to eliminate ambiguity in the stresses of the trusses, to reduce the effect of secondary stresses, and to secure increased stiffness as well as strength in the structure. Double systems of webbing have been practically abandoned so far as new construction is concerned. The simplicity of truss action thus secured permits the stresses to be computed with greater accuracy and thereby tends to economy. Before the last decade very few through bridges and those only of large span were designed with end floor beams in order to make the superstructure as complete as possible in itself and independent of the masonry supports. Now this improved feature is being extended to bridges of small spans. Similarly dropping the ends of all floor beams in through bridges so as to clear the lower chord and to enable the lower lateral system to be connected without producing an excessive bending movement in the posts has likewise been extended to the smaller spans of pin bridges and is now the standard practice. The expansion bearings have been made more effective by the use of larger rollers, and of bed plates so designed as to properly distribute the large loads upon the masonry. In the large spans of through bridges the top chord is curved more uniformly, thereby improving the esthetic appearance. These chords are also given full pin bearings, thus reducing the secondary stresses.

The stiffness of truss bridges has been secured by adopting stiff bracing in the lateral systems and sway bracing, instead of the light adjustable rods formerly used. At the same time adjustable counter ties in the trusses are being replaced in recent years by stiff ones, while in some cases the counters are omitted and the main diagonals designed to take both tension and compression.

Some of the same influences referred to above have led to much simpler designs for the portal bracing by using a few members of adequate strength and stiffness similar in general character to those of the trusses.

Such steady progress in the design and construction of railroad bridges of moderate span has, unfortunately, no adequate counterpart in highway bridges. The conditions under which highway bridges are purchased by township and county commissioners are decidedly unfavorable to material improvements in the character of their details. It is a comparatively rare occurrence that the commissioners employ a bridge engineer to look after the interests of the taxpayers by providing suitable specifications, making the design, inspecting the material, and examining the construction of the bridge to see that it conforms to all the imposed requirements. These provisions are only made in some of the cities, and accordingly one must examine the new bridges in cities to learn what progress is making in highway bridge building.

The lack of proper supervision in the rural districts and many of the smaller cities results in the continued use of short trusses with slender members built up of thin plates and shapes, whose comparatively light weight causes excessive vibration and consequent wear, as well as deterioration from rust. Under better administration plate girders would be substituted for such light trusses, making both a stiffer structure and one more easily protected by paint. The general lack of inspection and the consequent failure to protect highway bridges by regular repainting will materially shorten their life and thereby increase the financial burden to replace them by new structures. Some

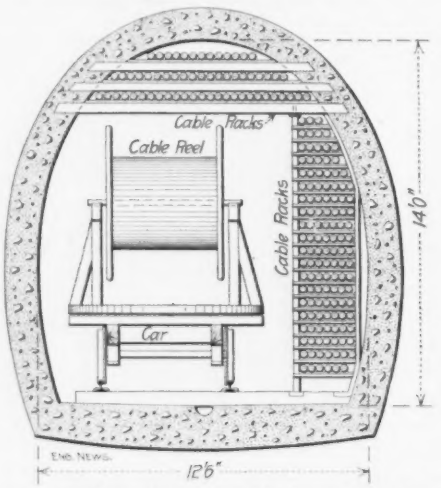

Fig. 1. Section of Large Telephone Tunnels in Chicago; Illinois Telephone & Telegraph Co. Illinois Telephone Construction Co., Contractors.

progress has been made in adopting riveted trusses for the shorter spans for which pin-connected trusses were formerly used, but the extent of this change is by no means as extensive as it should be, nor equal to the corresponding advance in railroad bridges.

The channel span of the cantilever bridge over the Mississippi River at Memphis, Tenn., is the longest one of any bridge of this class in America. It measures 790½ ft. between centers of supports. This bridge was finished in 1890, the same year that marked the close of the seven-year period of construction and erection of the mammoth cantilever bridge over the Firth of Forth in Scotland. A number of cantilever bridges have been built since then, but most of them have comparatively short spans. There is one now under construction over the Monongahela River in Pittsburg, which is expected to be finished this year, whose span is to be a little longer than that of the Memphis bridge. It is on the new extension of the Wabash R. R. system, and the distance between pier centers is 812 ft.

But there is another one being built which will not only have a longer span than any other cantilever bridge in this country, but longer than that of any other bridge whatsoever. It is located near Quebec, Canada, and its channel span over the Saint Lawrence River is to have the unprecedented length of 1,800 ft. or nearly a hundred feet longer than that of the Forth cantilever bridge and 200 ft. longer than the Brooklyn suspension bridge. The towers will have a height of 360 ft. above high tide. It will accommodate a double-track railroad, besides two electric railway tracks and highways. In the piers the courses of masonry are 4 ft. high and individual stones weigh about 15 tons each. The character of its design and the simplicity of its details will permit its construction with unusual rapidity and economy for a bridge of this magnitude. Several other cantilever structures whose largest spans range from 600 to 671 ft. are either now or soon will be under construction.

The Brooklyn bridge, completed in 1883, is still the largest suspension bridge in the world, its span being 1,595½ ft. More people cross this bridge than any other in any country. The New East River bridge, which is now being built, has a span of 1,600 ft., and its capacity will be very much greater than that of the Brooklyn bridge. Each of its four cables has a safe strength of over 30,000,000 lbs. in tension.

One of the most interesting developments relating to the subject under consideration is the construction of a considerable number of metallic arch bridges in recent years and the promise of their still greater use in the future. On account of their form they constitute one of the handsomest classes of bridges.

The past decade witnessed the introduction and extensive development of arches of concrete and of concrete-steel construction. In the latter kind a small amount of steel is imbedded in the concrete in order to resist the tensile stresses that may be developed. During this period more than 150 concrete-steel bridges have been built in this country. In the same year in which the largest metallic arch was completed, the five concrete-steel arches of the bridge at Topeka, Kansas, were finished. The largest one has a span of 125 ft., and still remains the largest span of this type in America, although it has been exceeded in Europe. Considerably larger spans are to be built this season, while others are included in the accepted design for the proposed Memorial bridge at Washington.

It is the smaller steel structures which are destined more and more to be replaced by arches of this material. The steel bridges require repainting at frequent intervals, constant inspection, occasional repairs and finally replacing by a new structure after a relatively short life, on account of rust and wear, unless it is required even sooner on account of a considerable increase in the live load. The concrete arch requires practically no attention except at very long intervals.

The Pennsylvania R. R. built a stone bridge, consisting of 48 segmental arches of 70-ft. span, at the crossing of the Susquehanna River at Rockville, Pa. It is 52 ft. wide, accommodates four tracks and cost $1,000,000. This bridge has not only the advantage of almost entirely eliminating the cost of maintenance, but it also has sufficient mass to withstand the floods which occasionally wreck the other bridges on that river. This year the same railroad is building a similar bridge over the Raritan River at New Brunswick, N. J.

Of movable bridges the largest swing span existing was erected in 1893 at Omaha over the Missouri River. Two years later a four-track railroad swing bridge was built by the New York Central R. R. over the Harlem River in New York city, which is only 389 ft. long between centers of end pins, but which weighs about 2,500 tons and is accordingly the heaviest drawbridge of any class in the world..

A bridge is being built across the Charles River between Boston and Cambridge that deserves especial mention and marks a decided advance in the growing recognition on the part of municipal authorities of the importance of esthetic considerations in the design of public works. It consists of 11 spans of steel arches whose lengths range from 101½ to 188½ ft. Its width is 105 ft. between railings. It is claimed that this bridge "will be not only one of the finest structures of its kind in this country, but will be a rival of any in the old world." Its length between abutments is 1,767½ ft., and it is estimated to cost about $2,500,000.

TELEPHONE TUNNELS AND STREET RAILWAY SUBWAYS IN CHICAGO.

The network of low-level telephone tunnels now being constructed under the streets of Chicago by the Illinois Telephone & Telegraph Co. was described and illustrated in our issue of May 2, 1901, and since that time considerable progress has been made. Some of the more recent tunnels, however, are considerably larger than the 6 × 7½ ft. section shown in the article above referred to, and the impression got abroad that the company intended to eventually operate a passenger service of some kind. In fact, several daily newspapers published long descriptions of the work, in which the President of the company, Mr. Wheeler, was reported as avowing an intention to carry mails and newspapers in cars between the post offices, railway stations and newspaper offices; the possibility of passenger service was also hinted at. This aroused the City Council, which made an investigation, in the course of which the company is said to have denied that it intended making any other use of its tunnel. The city authorities, however, arrived at the conclusion that the tunnels were larger than required by telephone requirements, and after prolonged negotiations, required the company to enter into an agreement under which it was necessary to apply for a new ordinance if the tunnels are to be used for any other purpose than the stringing of telephone and telegraph wires and cables. Fig. 1 is a section of one of the large tunnels, showing the space occupied by the cable racks and that left available for running a cable reel through the tunnel on a standard gage track.

Under the provisions of an ordinance drawn up

by the company, and modified by the Council Committee on Local Transportation, the tunnels may be used for the transmission of newspapers, parcels and mail matter, but no passenger transportation is allowed, and space must always be provided for the city's telephone, electric light and police and fire alarm wires. The Illinois Telephone & Telegraph Co., which owns the tunnels and the new telephone system, is to pay to the city 5% of its gross annual receipts for the first

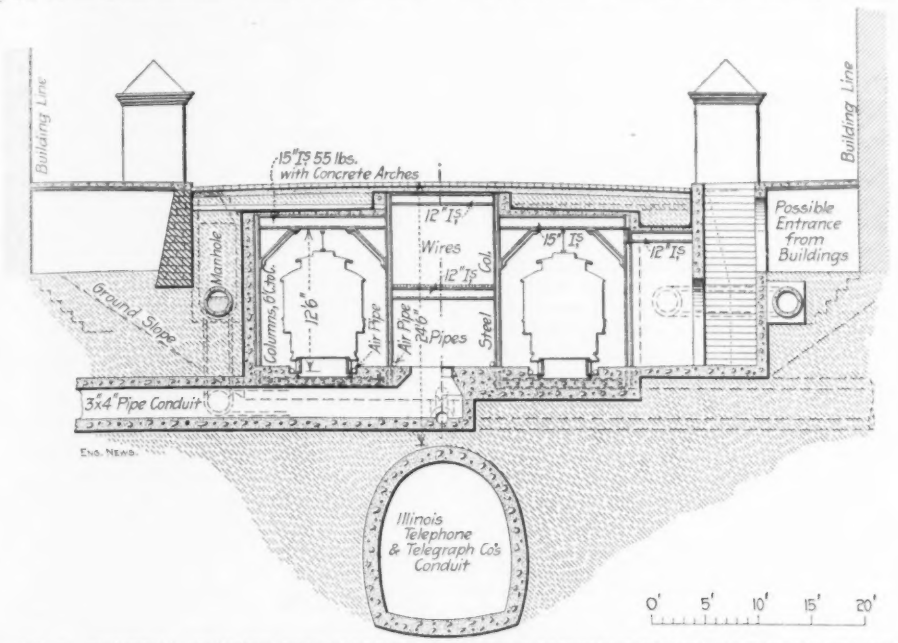

FIG. 2. GENERAL SCHEME FOR CHICAGO STREET RAILWAY SUBWAY; PREPARED BY MR. ERICSON, CITY ENGINEER, IN 1900.

ten years, 8% for the second ten years, and 12% for the remaining seven years, the franchise being for a term of 27 years. These rates were fixed in a conference between the company and the committee, but are subject to modification before the city council grants the franchise. The right is retained on behalf of the city to terminate the franchise at the end of 15 years, and to acquire the property at a fair cash valuation. This purchase would not include the company's telephone business. The telephone rates are fixed by the ordinance of Feb. 20, 1899, but the rates to be charged for other service will be subject to regulation by the city. The company is required to make annual reports to the city, and cannot sell or transfer its franchise rights without the consent of the city.

The public opinion that these tunnels were intended to be used for traffic purposes was strengthened by an address delivered before the Chicago Real Estate Board by Mr. George W. Jackson, manager and engineer of the construction company which is building the conduits. In this he outlined a plan for an underground line for street cars, but he advocated a line as near the surface as possible, in order to avoid interference with the foundations of large buildings, while the tops of the telephone tunnels are about 26 ft. below the streets. The excavation for a large subway at any such depth, with its stations and passages to the surface, would seriously endanger the foundations of buildings.

A comprehensive plan for a subway system was devised some time ago by Mr. John Ericson, City Engineer, as shown in Fig. 2. This was somewhat on the lines of the Boston subway, but he believed that it would be almost impracticable to build such a line on account of the space required by the network of gas and water mains; electric conduits, etc., with their laterals, manholes, etc. Mr. Jackson stated that the streets in the downtown district are virtually occupied from curb to curb for a depth of 9 ft., while the street intersections are occupied by the manholes of the different corporations. He thought, however, that some relief might be afforded if the engineers of all these concerns would confer together. His plan provided for single track loop terminals for the different lines of street railway, practically duplicating their present surface loops. This arrangement, however, would have the effect of perpetuating the down-town terminals, while the present movement is strongly towards obliging the railway companies to run cars through the city instead of concentrating them all at terminals in the congested district, involving change of cars and double fare for a through trip from one side of the city to the other. He admitted, however, that a double track subway would be more satisfactory, in view of the growth of traffic. Fig. 3 shows sections of Mr. Jackson's proposed single track subway.

Two underground railway schemes are now being promoted in Chicago. A syndicate represented by Mr. Hempstead Washburn (a former mayor) and Mr. W. A. Alexander, intends to ask for a franchise to build a subway system for transportation, gas and sewer pipes, water mains and electric conduits on about seven miles of streets in the business section of the city. The cost is estimated at $35,000,000, and the representatives claim that this is all promised, though they refuse to say by whom. The subway would extend from curb to curb, and its bottom would be about 24 ft. below the street surface; thus the telephone tunnels now under construction would not be interfered with. The north and west entrances

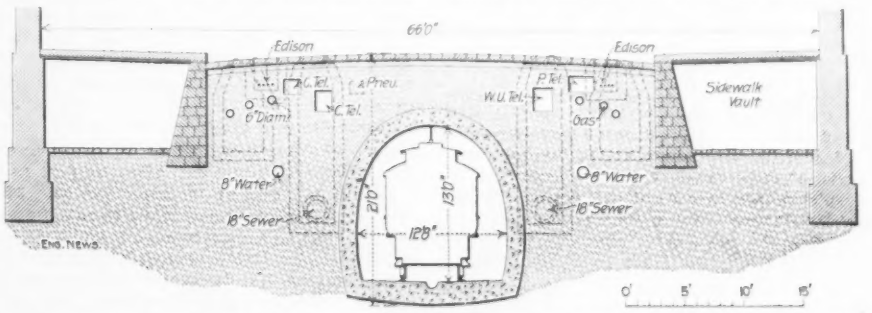

FIG. 3. SECTIONS OF SINGLE-TRACK LOOP SUBWAYS FOR CHICAGO; PROPOSED BY MR. GEO. W. JACKSON.

would be as close as possible to the north and west banks of the river, while the south entrance would be at Harrison St., though the subway would eventually be extended to 12th or 16th St. The city engineer has undertaken an investigation of the effect of such a subway upon the foundations of the tall buildings. The second project is to take up the old franchise of the Chicago Steam Heating & Smoke Abating Co. (granted under what was known as the Bacon ordinance in 1892), and the similar franchise granted later to the Chicago Light, Heat & Power Co. The former company laid about three miles of pipe from the Fraser & Chalmers works in Lake St., and passed steam through the pipe occasionally in order to hold the franchise, but whether this old franchise has now any authority is held to be a matter of doubt. No franchise has yet been granted for either of the projects above noted, and when franchises are applied for for these or any similar projects it is likely that the main point of discussion will be the terms for compensation to be paid the city and the rights of the city to acquire the property, as there is in Chicago an encouraging growth of popular opinion in opposition to the granting of valuable franchises without requiring any adequate compensation or return for the city.

THE DUTIES OF ENGINEERS IN ENFORCING CONTRACTS.

By Albert J. Himes,* M. Am. Soc. C. E.

In the last half-dozen years the conduct of engineers in the enforcement of contracts for public works has been a frequent matter for public criticism. Moreover, so many conflicting opinions have appeared concerning it from engineers themselves that the professional mind appears to be quite hazy on this very important matter. A thorough discussion of the subject may clear up many doubts and misconceptions and make life happier for those charged with important trusts who are a little lacking in experience. The writer has felt the need of a safe criterion in such matters, and in submitting what follows he hopes to profit from the remarks of others whose experience has been wider than his own.

In the first place, it is noticeable that the engineer is seldom made the executive head of a great undertaking. His position is rather that of an expert counselor on subjects within his sphere. In early life he is a surveyor or skilled workman, his authority exists because of certain powers to calculate and plan which untrained men cannot readily acquire, but in matters of inspection he is set aside as being too young or impractical. A very prominent engineer once remarked that "the engineer could not control the quality of construction, that one must depend on the inspectors." That was a very practical statement, as it expressed a general condition, but it does not prove that the condition is a necessary one.

What can be more unjust and discouraging than that an engineer should be censured for the failure of his plans when the real cause of the failure is their improper execution by parties not responsible to the engineer.

It is told of the late Professor Gillespie that he was asked by Horace Greeley to design a dam of a certain height. He did so and the matter was forgotten. Some time later there appeared in Greeley's paper a fierce denunciation of the engineering profession, and Professor Gillespie, on inquiry, learned that Greeley had built a dam on his estate and the dam had failed. A more thorough investigation showed that after building the dam according to the plan, a considerable addition was made to its height, but the section remained unchanged. Failure was inevitable.

The present-day editor would be no more considerate if through the inefficiency of inspectors appointed by some Board of Public Works a dam should fail which had been correctly designed but improperly drained and cemented.

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It is now becoming generally recognized, not only by engineers but by well-informed business men, that any large piece of construction is best performed by a competent engineer acting as the executive head of the undertaking. Such a man is, of course, dependent on inspectors, but they are of his own choosing and so are his engineering assistants.

When the nature of the work requires the use of skilled labor, a good mechanic is often the best available inspector, but the writer's experience would indicate that with necessary preliminary training an engineering graduate makes the best inspector obtainable, and that the more general employment of such men would make an improvement in the quality of engineering work.

To insure good work it is necessary for the inspector to feel that he has the active sympathy and good will of his employer. There are many critical occasions in the progress of a work where it is difficult to verify statements made by either the foreman or the inspector, and if there is a tendency to ignore or overrule the inspector he is not likely to incur the ill will of the workmen and the rebuke of his employer by insisting that work shall be done right when the errors are quickly hidden and may never be known. The proper way is for the employer to visit his inspectors frequently, to be familiar with their work and give them necessary instructions without waiting for an appeal from the contractor. Such an appeal is generally the result of long-standing tension between the inspectors and foremen, and if the inspector is overruled, he might as well be relieved. His usefulness on that work, if not wholly gone, is greatly lessened.

There is a tendency among engineers who have a limited experience in construction to be superficial in their inspection. To use their language:

> It is inadmissible to pry into the contractors affairs or exhibit suspicion of his work, such treatment will drive any man to crime.

But the engineer who takes such a view of this matter writes himself down as unfitted to take responsible charge of work. Such a man would not ask for competitive bids for merchandise as required in government affairs, as such a course would imply a suspicion that the first merchant consulted was unfair; he would not ask for a receipt for money paid, as it would imply a suspicion that the payee would later deny the payment. In short, such a man has too sensitive a nature to do business. He ought to play golf.

It is never dishonorable to ask a man to perform his contract, and no reputable contractor will refuse to do so if it is insisted upon. The production of bills of material and receipts for payments of claims that may become liens on the work are ordinary safeguards, and in no sense a persecution or an insult.

In matters of business a man knows only what he sees, and for an engineer to say he knows a certain thing has been done because the contractor said he did it is conclusive evidence of a weak nature.

It is no injustice to contractors to say that they need watching. In fact, some men like to be watched, and like to know that the inspector understands what he sees. Efficient inspection allays suspicion, and in case of unforeseen difficulties it is much easier for a contractor to get assistance if some one knows that his work has been well done. On the other hand, an officious and incompetent inspector is the bane of a contractor's life.

Specifications are frequently written by some one more or less unfamiliar with the practical methods of doing work, or worse than that, they are inherited and are out of date. In such cases interpretation is necessary, and permission should be given in the contract for such interpretation.

The idea of reducing the working parts of a large corporation to a standard is a good one, and should result in economy, but when the standardization is dependent upon an inflexible order that no departures be made, much confusion and loss will follow.

In the interpretation of contracts a wide experience in general business, as well as construction, and a full knowledge of existing conditions are necessary for success. Perfection does not exist, and it is even rare that men are willing to strive for it. Business men, experienced and successful in one particular line, are prone to look with contempt upon the efforts of an engineer to secure perfect work. Their expenses for handling, storage, breakage, rent, help and the like are so variable that large margins are necessary. Thirty, fifty, and even one hundred per cent. increase in selling price over purchase price are not uncommon, and judging by their own affairs they doubt the wisdom of insisting on the least particular of a contract.

Again, there has been so much competition in engineering work, and the profits have been so small, that the enforcement of minor requirements has seemed like nagging.

The subject is not purely an engineering matter, but largely one of business and human nature. The primary aim of each party to a contract is to get all he can out of it and their methods are governed by expediency. Where the amount involved is small and a continuance of business relations is desired—as in the case of retail trade—the satisfaction of the purchaser is of prime importance. When a railroad has a large amount of work to do, a contractor is interested to give satisfaction in order that he may receive more work. If the company is doing a legitimate business, it is likewise interested to have the contractor, if he is a good one, make enough profit to keep him interested in the work. Where the contract is large and the parties thereto will have no further business relations, the aim of each is to get all he can and still effect a settlement without a suit. That is the contractor's position in a public works contract.

Such are the conflicting interests of the principal parties, and the engineer holds a very peculiar position. He is in the employ of one of the parties, but must nevertheless be an impartial arbiter between the two.

There seems to be some confusion of ideas as to the rights and duties for engineers in private and public work, respectively.

In private business, the specifications of an engineer are looked upon as merely one of his tools. They are seldom understood, or appreciated, or even examined by the parties who pay for the work. The final judgment is passed upon the results, regardless of the manner of achievement. The authority of the parties furnishing the money is final, and their satisfaction is the engineer's success.

Not so in public affairs. The final judgment is then passed by the people, and it is not enough that the best posted or even that the majority should be satisfied. Any departure from the strict letter of the contract is treated as an infringement of the rights of the minority and will be punished.

Unsuccessful bidders, their friends, politicians and adventurers will make capital out of any departure from the straight and narrow path. Such a departure is ammunition, and it will be used. The weight of a name and a reputation cannot curb the yellow journals nor muzzle the malcontents.

Men who are foremost to condemn an official for improper conduct will often be first to try to influence his acts by improper means. The law condemns the acceptance by an agent of fees from both parties to a transaction if the facts are unsanctioned by the principals. Yet all kinds of valuable presents, discounts, etc., are offered by reputable business men to cultivate the good will and patronage of public men.

It is therefore wise that every government contract should be drawn with great care to avoid the chance for disputes over its interpretation.

A contract must be complete and definite in all its terms, and a contract to do impossibilities is void. Material modifications of a contract may amount to an abrogation thereof. Large contracting firms employ lawyers, and are better posted than engineers upon the fine points of contract law. They often seek to do extra work or to change the plans or in some way to get away from the original contract that they may have a claim for adjustment, a claim whose accuracy no one can prove, and which could not exist if the original contract were properly drawn and enforced, but a claim which is often settled at large cost to avoid prolonged disputes.

It is doubtful if any authority can be legally exercised to vary the terms of a contract entered into as a result of competitive bidding in pursuance of an enactment of law.

THE METALLURGY OF THE CUPOLA.*

By H. E. Field.†

The quality of cast iron depends, to a great extent, upon cupola practice. The best of pig iron can be changed into inferior cast iron by poor melting; while an inferior pig may, by judicious melting and mixing, be used in high-grade castings. There are hardly two cupolas in existence which work under exactly the same conditions, with the same-sized tuyeres; the same amount of blast, with equal height of tuyeres from cupola bottoms, etc. For every variation in cupola arrangement, there is a variation of the effect in melting iron. The greater the difference in the cupolas, the greater will be the variation in the melted product. Pig iron and coke merchants frequently find that two firms engaged in exactly the same line of work and using the same iron and coke, get very different results. When pig iron, in the course of melting, comes in contact with the blast, and is acted upon by it, certain of the metalloids are burned out of the iron. This is true in all cupolas. The amount burned out increases with the amount of air forced into the cupola by the blower. In the same cupola, and under the same conditions, irons varying in composition will lose different amounts of impurities. As the melted iron, freed to a certain extent from some of the metalloids, passes through the incandescent bed of coke or coal, it absorbs new impurities from the bed. The amount absorbed varies with the cupola practice. From a heavy or deep bed, other conditions being equal, more impurities will be absorbed than from a light or shallow bed. The temperature of the iron at this time has a decided effect upon the amount of metalloids thus taken up by the iron, as does also the amount of flux used. With a hot running cupola and a large amount of flux, little sulphur will be absorbed by the iron, while a large amount will come off with the slag.

There has been much discussion in the last year or two as to whether certain of the metalloids lose or gain in passing through a cupola. The great variation in the results obtained by reliable experimenters has led to a general belief that there is no fixed rule which can be followed in figuring these cupola losses and gains. This idea, however, is incorrect, as I shall try to prove by taking up each metalloid separately.

SULPHUR.—Under ordinary condition, iron melting in a cupola takes up sulphur from the fuel. The greater the percentage of fuel and the greater the percentage of sulphur in the fuel, the greater will be the amount of sulphur absorbed by the molten iron. The ratio between the total amount of sulphur present in the fuel and the amount absorbed by the iron is dependent upon three conditions: First, the kind and quality of flux used; Second, the temperature of the iron; and third, the composition of the coke and iron. A proper quantity of flux in a hot working cupola will take care of a considerable amount of sulphur. The sulphur present in the fuel as a sulphuretted hydro-carbon has no appreciable effect in increasing the percentage of sulphur in the melted iron. This accounts for the fact that many foundries melting with coal obtain castings with a lower percentage of sulphur in proportion to the amount of sulphur in the fuel than do foundries melting with coke. The greater the amount of manganese in the iron, the less sulphur will the iron absorb; and it is possible in cases of very high manganese iron for that iron to lose sulphur in melting; the sulphur passing off as manganese sulphide with the slag.

SILICON.—In passing through a cupola iron always loses silicon. The amount of the loss depends upon two conditions: First, upon the amount of oxygen brought in contact with the metal in melting, and Second, upon the composition of the iron as it goes into the cupola. In giving the amount of silicon lost in passing through a cupola, authorities generally mention a fixed amount. This may have been true concerning the grade of iron with which the writers were familiar. But it would be wholly untrue concerning other grades. The greater the amount of blast, the more silicon will be burned out. The higher the percentage of silicon, the greater will be the percentage of loss in passing through a cupola. (By percentage of loss, I mean ratio of lost silicon to original amount of silicon in the pig.) An iron running 4% in silicon, will lose up to 20% of its original amount, while the loss in an iron running 0.2% is hardly perceptible. This variation in the loss of silicon is accounted for by the fact that the lower the percentage of silicon in the iron, the greater is the affinity of the iron for that silicon, and the more difficult will it be to oxidize it; while on the other hand, the greater the amount of silicon present in the iron, the less will be the affinity and the greater the amount oxidized in melting. The percentage of carbon in the iron has a great deal to do with the loss of silicon in melting.

*Extracts from a paper read at the annual meeting of the American Foundrymen's Association.
†Ansonia, Conn.

This, however, will be taken up later under the consideration of carbon.

PHOSPHORUS.—A very slight amount of phosphorus is absorbed from fuel by iron in melting. It is practically, however, a constant quantity, the small increase being proportional to the amount of phosphorus in the fuel.

MANGANESE.—Manganese loses in passing through a cupola. Its action is that of a protector from oxidation and the amount lost in passing through a cupola depends upon the amount of blast and also upon the percentage of sulphur in the fuel. The greater the amount of air forced into the molten metal the more manganese there is lost as an oxide. The greater the percentage of sulphur in the fuel, the greater will be the percentage of manganese passing into the slag as a sulphide. Manganese is not united with iron, as a compound, it is rather alloyed with it, having practically no affinity for the iron.

CARBON.—I have left this element until the last for several reasons. Principally, however, because its action is understood less than that of the other elements, which we have to consider in connection with melting iron.

Iron may both gain and lose in carbon in passing through a cupola, and whether the gain or loss predominates depends, first upon the original composition of the iron, and second, upon the conditions or method of melting. An iron while passing down the cupola has a portion of its carbon burned off, the quantity depending upon the amount of blast and the time which the iron consumes in passing through that zone of the cupola, where the oxidizing influences are at work. After having a portion of its carbon removed, the molten iron comes in contact with the incandescent coke in the bed, and from this it absorbs carbon. The hotter the iron, the larger the amount of fuel, and the longer the iron remains in contact with this fuel, the more carbon will it absorb. If the amount of fuel is sufficient, the iron may absorb more carbon than it loses in passing the tuyeres, and thus the iron will have gained in carbon. If, however, the blast is heavy and the ratio of fuel to iron small, the iron will lose more carbon in passing through the oxidizing zone than it can possibly pick up from the bed, and thus the iron loses in carbon. These two conditions are at work every day in our cupolas. Melting with low blast and high percentage of coke will cause a gain, in carbon, while the high blast and low percentage of coke will cause a loss of carbon.

The original composition of the pig iron will have much to do with the loss or gain in carbon. An iron made at the blast furnace with a large percentage of fuel will be high in carbon. This if melted in a cupola with a small amount of fuel, will lose in carbon. While a pig iron low in carbon, made by a blast furnace using a small allowance of fuel, will, when remelted in a cupola with a relatively large amount of coke, gain in carbon. A low carbon iron, then, will tend to gain in carbon, while a high carbon iron will tend to lose in carbon in melting in a cupola.

There is one more factor which we must consider in discussing this loss or gain of carbon in cupola practice, and that is the effect or rather the relation of silicon to carbon in this process. I have said that low carbon iron gained in carbon. Let us now qualify this statement by saying that a low carbon, low silicon iron, will gain in carbon. Carbon and silicon, although they are analogous elements in many of their properties, have always been considered to be almost opposite in their action on cast iron. The facts of the case are, that their actions are almost identical although the results are different. Pure iron we know will absorb carbon up to 6.67%, while it will absorb silicon up to 23%. One part of carbon then is about 3½ times as effective on cast iron as is one part of silicon. We find this ratio to exist in our pig and cast iron to a marked degree. For every rise of 0.1% in carbon in our pig iron made under the same conditions, there will be a corresponding decrease of 0.35% in silicon, and vice versa. This applies in the same manner to our cupolas. When we melt a pig of say 3% total carbon and 1% silicon, we would expect under ordinary conditions a gain in carbon. If, however, we melt a pig of 3% carbon and 4% silicon, we are sure, under the same conditions, to find a loss in carbon. Iron in the melted state may be thought of as a solution in which carbon and silicon each reduces the power of the iron to dissolve the other.

We have found then, that a small percentage of fuel, high blast, high carbon, and high silicon tend to cause iron to lose in carbon, while low blast, large percentage of fuel, low carbon and low silicon tend to cause iron to gain in carbon in melting.

When we consider the great variety of iron in use in foundry practice, and the divergency in cupolas, we can readily understand that no hard and fast rule can be laid down for all cases. Every foundryman must find out for himself, in his own cupola, and with the brands of iron which he uses, or else he must obtain his knowledge from some one whose experience is similar to his own.

Are our cupolas so arranged as to produce at the least cost, the best iron for our work? Are we, in order to save a few cents, running them with the least fuel possible, and paying out at the same time, dollars to obtain the superior grades of iron necessary when iron is melted with little fuel? Are we melting with heavy percentages of fuel, and then adding limestone to reduce sulphur, and steel to reduce carbon? Are we using high silicon to soften, when it is really high carbon that is needed, or high carbon when it may be that high silicon would answer the purpose very much better?

For various kinds of work, different grades of iron are required; stove plate and the like demand fluidity and softness, and to secure these an iron high in silicon, high in phosphorus, and rather low in carbon and manganese is most used. For light machinery, an iron high in silicon, medium in carbon and phosphorus and low in sulphur and manganese is necessary. Medium machinery requires an iron medium in silicon and carbon, low in phosphorus and manganese. In heavy machinery where weight rather than strength is desired, and where castings must be free from shrinks and segregations, iron low in silicon and phosphorus, high in carbon and manganese is essential, and a somewhat higher percentage of sulphur may be permitted. In castings where a high strength is desired, an iron low in silicon, low in carbon, low in phosphorus, low in sulphur, and medium in manganese is necessary. Cupolas then, should be so constructed as to best melt an iron suited for grades of its particular foundry.

The height of the tuyeres must regulate the manner in which the iron is taken from the cupola. Low tuyeres necessitate that the iron be drawn from the cupola as fast as it is melted, and either be held in a reservoir or taken away in small ladles. High tuyeres allow the iron to be collected in the hearth of the cupola and to be drawn off in large taps, and have the advantage of allowing crane ladles to be shifted while the iron is collecting in the cupola. When low tuyeres are used for filling crane ladles, if any time is apt to be lost in changing the ladles, two spouts should be used, one ladle being placed in position while the other is filling. We have mentioned that for stove plate and like grades of work, a high silicon, low carbon, low sulphur and low manganese iron is best suited. We have also mentioned that the amount of carbon and sulphur which an iron will absorb is dependent upon the amount of fuel which it passes through. Hence, for stove plate, and like work, the lower the tuyeres provided they are consistent with good melting, the better suited will the iron be for the work. This does not interfere with the general practice in stove plate shops of removing the iron in hand ladles and by trolley tracks as fast as it is melted.

Light and medium machinery castings require only a medium carbon percentage, and are much more advantageously melted in low tuyere cupolas. The somewhat lower silicon used in this line of work furnishes a higher carbon than is present in stove plate iron, and the small bed gives a low sulphur percentage. In the heavy machinery where softness and soundness are required, a large bed has an advantage. Low silicon iron which is cheaper may be used, and the carbon absorbed from the deep bed makes an iron high in carbon, while the resulting high percentage of sulphur is less detrimental in this heavy grade of work. The deep hearth allows the collection of a large amount of iron and thus makes the custom of tapping into crane ladles practicable.

In the so-called strong irons, named by some gun iron by others semi-steel, we have a somewhat more complicated problem. These irons must, first of all, be low in carbon, and low in sulphur, and it is just such irons as these that absorb most readily carbon and sulphur from the fuel. There are cupolas in operation in many of our foundries from which it is practically impossible to obtain an iron above 28,000 lbs. tensile strength for the reason that the height of the tuyeres requires so great a depth of fuel in the bed, that the iron will absorb an amount of carbon and sulphur so great that it is impossible to get a strong casting. For the melting of this grade of iron, low tuyeres are essential, and, of course, the iron must be removed from the cupola as fast as it is melted. The majority of cupolas in operation to-day have extremely high tuyeres, and we have seen that these are advantageous for but a single grade of work.

In advocating the lower tuyeres for cupolas I have always met with the above-mentioned objection in regard to changing the ladles in time to prevent the iron getting into the tuyeres. It is to obviate this difficulty that I suggest the double spout on cupolas which are so arranged as to make this possible. The modern tendency in building cupolas is to make the tuyeres as low as possible in order to show a high fuel ratio, and I venture to predict that they will be built much lower as soon as the double spout arrangement is more universally adopted.

THE GYPSUM PRODUCTION of the United States in 1901, says Dr. Joseph Struthers, in "Mineral Resources of the U. S. 1901," now in press, amounted to 659,659 short tons, valued at $1,577,493. This is an increase of 65,197 tons in quantity, and a decrease of $45,710 in value over the output of 1900. Of the 1901 production 65,698 tons were ground into land plaster, and 521,292 tons were made into plaster of paris. In the order of production Iowa, Kansas and Texas come first, followed by Michigan and New York.

THE STONE PRODUCTION of the United States in 1901, says the above work, aggregated in value $60,982,060, a gain of $12,295,405 over the total for 1900. Limestone shows the greatest increase, with a total in value of $26,406,897; granite comes next, with $15,000,000; the sandstone output was valued at $9,000,000. In the order of production in every kind of stone Pennsylvania ranks first with a total value of $10,772,288; Vermont is second, with $5,366,740; Ohio next, with $5,183,225, and then come New York, Maine, Massachusetts and Illinois in the order named.

THE COPPER MINE OF CERRO DE PASCO, PERU, said to be the largest single deposit of copper in the world, is being developed by James B. Haggin, of New York, and his associates. These mines have been famous as silver producers for more than 250 years, and were worked by the Incas before that. Later investigations show enormous copper deposits, and $3,000,000 is said to have been already expended by the present syndicate in securing about four-fifths of the entire mineral region. The mines are located 80 miles from Oroya, and a railway is to be built at a cost of $2,000,000, connecting them with the Oroya Railway, terminating at the port of Calbao. This road will be standard gage and cross a ridge of the Andes at an elevation of 14,000 ft. With the exception of two short lengths of 3%, the average grade will be 1½%.

PETROLEUM PRODUCTION IN THE UNITED States, in 1901, is treated by Mr. F. H. Oliphant, in "Mineral Resources of the United States," now in press. The U. S. Geological Survey, in a press bulletin, summarizes this report. The petroleum production was greater than for any previous year, amounting to 69,389,194 barrels of crude oil, valued at $66,417,335. This value shows a decline from 1900, when 63,620,529 barrels were valued at $75,989,313. In the order of production Ohio comes first with over 21,000,000 barrels; then West Virginia, with over 14,000,000 barrels; Pennsylvania, over 12,000,000; California, with over 8,000,000; Indiana, over 5,000,000; and Texas sixth, with over 4,000,000 barrels. The average price paid for all petroleum marketed in the United States in 1901 was 95.7 cts. per barrel, as compared with $1.19 4-10 in 1900. But the average price for Pennsylvania oil per barrel was $1.21. In 1901 14,250 wells were completed, with 3,220 estimated to be dry. The cost of these 14,250 wells is estimated at $21,375,000, or about one-third of the value of the total crude product for that year. In 1901 1,062,750,306 gallons of petroleum and its derivatives was exported, valued at $71,479,124; the average price paid for this exported oil was 6.73 cts. per gallon, as compared with 7.52 cts. received in 1900.

THE GRAPHITE INDUSTRY IN THE UNITED States for 1901 is described by Dr. Joseph Struthers in the "Mineral Resources of the United States," now in press. The output of crystalline graphite was 3,967,612 lbs., valued at $135,914, and this came chiefly from the mines at Ticonderoga, N. Y. Of amorphous graphite 809 long tons were produced, valued at $31,800; this includes the so-called graphitic anthracite of Rhode Island and the Baraga graphite of Michigan. In 1901 14,320 long tons of graphite were imported in the United States, valued at $895,375, the higher grades coming from Ceylon. Canada produced 2,190 tons in the year, mainly from the Brougham mines, in Renfrew county, Ontario. The production of artificial graphite has largely increased in this country, the output for the last year amounting to 2,500,000 lbs., valued at $119,000; in 1897 we only made 162,382 lbs. The sole producer of this artificial graphite is the International Acheson Graphite Co., of Niagara Falls, N. Y. More than half of the output was in the form of graphitized electrodes for use in electrolytic processes; the remainder is consumed in the manufacture of paints, lubricants, pencils, motor brushes, crucibles and dry batteries.

THE STEEL AND IRON PRODUCTION of the United States is treated in a late issue of the London "Commercial Intelligence," says the U. S. Bureau of Statistics. According to this journal the world's total product of pig iron, in 1901, amounted to 40,408,000 tons. Of this the United States contributed 15,878,000 tons; the United Kingdom 7,750,000 tons; Germany, 7,633,000 tons; Russia, 3,100,000 tons, and France, 2,362,000 tons. As compared with the annual average for the period 1866-70, the 1901 product for the United States shows a gain of 985%; that for the United Kingdom 51%, for Germany 525%, and for the entire world exclusive of these mentioned, 236%. In the period 1866-70 the world's per capita consumption of pig iron was 17 lbs.; in 1901 it was 57 lbs. And, while in the latter year the United States consumed 455 lbs. per capita, the United Kingdom used 350 lbs. In 1882 imports of iron and steel into the United States formed 9.3% of total imports and were valued at $67,976,897; in 1901 these imports formed only 2.2% of the total and were valued at $17,874,789. On the other hand, in the same period our exports of iron and steel have grown from $20,748,206 to $117,319,320, or from 15% to 28% of exported manufactures.

THE MANAGUA-CORINTO RAILWAY, in Nicaragua, has been completed, shortening the time of transit between these points four hours. This link obviates the necessity of making part of the journey by steamer on Lake Managua and transshipment at Momotombo, at the head of that lake. Two of the three steel screw steamers used in this lake traffic will be now transferred in sections to Lake Nicaragua and relaunched and used on the San Carlos-Grenada route.

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ADVERTISING RATES: 20 cents a line. "Want" and "For Sale" notices, special rates, see page XXIV-XXIX. New copy for standing advertisements should be received one week in advance of publication; new advertisements, Monday morning. The pages containing "Want," "For Sale" and "Proposal" advertisements are held open until Wednesday noon.

A promising line of work undertaken by the U. S. Geological Survey is a collection of chemical and bacterial analyses of water from various sections of the country. Not long ago we noted that the Division of Hydrography had issued a circular describing standards for color and turbidity, for the use of its observers. Circular No. 9, entitled "Water Analysis," has just been published by the same division. It states that many individuals and associations are rendering supplemental assistance, and that analyses are being made in local laboratories. The circular is designed to secure uniformity of methods and results, and to that end includes an outline of analytical methods and blank forms for reports. The methods of analysis recommended are those adopted by the committee on uniform analyses of the American Public Health Association, of which Mr. Geo. W. Fuller, Assoc. M. Am. Soc. C. E., 170 Broadway, New York city, is chairman, and Mr. Geo. C. Whipple, Assoc. M. Am. Soc. C. E., Mt. Prospect Laboratory, Brooklyn Water Department, is secretary. Mr. F. H. Newell, M. Am. Soc. C. E., is hydrographer in general charge of the water supply investigations of the Survey, which investigations are increasing in scope and usefulness, with the addition of such new lines of work as that just outlined.

The city waste problem in New York has impressed Mayor Low as being in a very unsatisfactory condition, but judging from some of his remarks on the subject it seems doubtful whether he really appreciates the nature of the problem. In the course of a recent letter approving opposition to the location proposed for a garbage dump Mayor Low said:

> I take advantage of this opportunity to say that the method of disposing of the city's waste by carrying it away by boats is a relic of barbarism. This waste ought, for the most part, to be burned; and the city should never be satisfied until the heat so generated is made a source of income to the city. It is no easy problem to bring about this result in the city of New York. It probably will take years fully to accomplish it, but I am confident that in time it can be accomplished if public sentiment will support every effort made in that direction.

The quotation shows that the difficulty of the problem is realized by the Mayor, but why is it any more barbaric to use boats to transport garbage to the final place of disposal than any other means of conveyance. Moreover, what evidence has the Mayor that under a proper system of ultimate disposal burning wastes would yield any more revenue than some other means of utilization? In fact, what evidence is there that any revenue can be secured by burning the mixed refuse of American cities? We do not mean to say that Mayor Low is wrong, but to point out how little is really known about the subject in question. If Mayor Low and his administration will address themselves to obtaining some comprehensive, unpartisan information on this subject, New York and all other American cities will be greatly benefited thereby. What is urgently needed is a thoroughgoing investigation, on a practical scale, and under the direction of competent engineers, of the whole subject of garbage and refuse disposal, and particularly the utilization of heat and the real truth about the possible revenues from garbage reduction processes. All the reduction plants thus far have been owned and operated by private companies, and little or no reliable information about their financial results is available.

A MUNICIPAL PROGRAM ESSENTIAL TO SYMMETRICAL AND CONTINUOUS CITY DEVELOPMENT.

The needs of most of our cities and towns are so far beyond the capacities of their municipal treasuries that some or all of the various departments are in a continual state of starvation. From every quarter comes a strong and persistent demand for a low tax rate, while on all sides an enormous pressure for various public services makes itself felt. Under such conditions the need for a well-considered program, conceived and executed in the interests of the city as a whole, is apparent; but in how few American cities can this be found? In most cases the various public services are controlled by separate and unrelated departments, each a rival of the other in the matter of securing appropriations. Practically the only time when the needs of the city as a whole are taken into account is when the annual appropriations are under consideration. Then each department is forced to struggle for its own needs, if not for life, while the body controlling the purse strings, acting under the demand for a low tax rate, is liable to cut closest where the protest will be weakest, rather than adjust all the appropriations to the relative needs involved.

The primary need in the majority of American cities is a more systematic method of dealing with annual appropriations—a method involving a complete budget, or preliminary estimate, of receipts and expenditures for the year under consideration. Of course, all cities make some such estimate, but in most cases it is very incomplete, and in some so crude as to be of little or no use as a means of apportioning the revenues to the several departments.

Given a board or an official competent to make such a budget, the next step in advance would be to formulate a municipal program, by which is meant a systematic plan for city development, designed to meet, in the order of their importance, all the wants of the city as rapidly as possible without incurring too great a financial burden. These wants have already become so numerous and varied, and are increasing so rapidly, that to give each proper weight is in itself no easy task. On this account, and because of the vital relations between a properly conceived municipal program and both public and private welfare, the work should be entrusted to none but the most competent and responsible persons in the community. Preferably, the framers of such a program would be the heads of the various municipal departments, and eventually the action of some or all of these would be necessary for the adoption of the program, but it would be both natural and highly advantageous to secure the advice and assistance of business and professional men of experience and standing in the community, both as an aid in the actual formation of the program and to give it weight before the public.

Among the several municipal departments engaged in framing a municipal program none would take a more active part than the engineering. This would follow: (1) Because of the many engineering and allied problems involved; (2) because engineers, as a rule, are more familiar with both the broad general phases and the details of municipal affairs than the members of any other profession; and (3) because their tenure of office, though all too uncertain, is comparatively long, thus contributing to the permanency of the policy laid down by any program. On (1), practically all will agree. Certainly it is only necessary to glance over the field of municipal activity to see how much of the work in hand is of an engineering character, or, at some stage, demands the services of an engineer. Probably the largest single item of municipal expenditure falls under the general head of education, but even here the engineer must assist if the best results in heating, ventilation and general sanitation are to be secured. Outside of education, certainly the bulk of municipal expenditure is for street construction and maintenance, water supply, sewerage, lighting and other public works of an engineering character. Regarding (2), the superior knowledge of municipal problems possessed by engineers, this follows naturally from (1), but aside from that, it is attested by the special training in several branches of municipal affairs given in the engineering schools, and by the number, age and strength of societies devoted wholly or in part to municipal engineering and allied topics. As to (3), longer tenure of office, there may be more doubt, but unquestionably city engineers remain undisturbed for longer periods than many other city officials, while the chances are that when not holding permanent positions they are doing general engineering work for one or more cities, and thus not only "keeping their hand in," but broadening their experience.

The scope of a municipal program is too large a subject to be discussed in detail at this time, but perhaps some idea of it may be conveyed in a few words. Its object is twofold: (1) To secure an orderly, systematic development of the city, instead of haphazard, unbalanced growth or general stagnation; (2) to so adjust expenditure to income as to avoid excessive taxation, and to so proportion the city revenues that one department will not starve while another fattens. To bring about the latter there must be an improvement, in most cities, in the budgetary system, as already indicated, and with it must come many financial reforms which cannot even be mentioned here. This, however, is in a way subsidiary to the municipal program, or perhaps a part of its execution, rather than its conception. To draw up the program will require such a broad and at the same time detailed survey of the status and needs of the municipality, as has been rarely if ever attempted. As a suggestion for a start in such a survey, the following heads may be mentioned: (1) The adequacy of the plan of the city, including the layout of its streets and public places from the viewpoints of intercommunication and civic impressiveness. (2) Pavements, sidewalks and bridges. (3) Transportation, including steam railway approaches, grade crossings and union or other depots, as well as street railway facilities. (4) Verbal communication, including the telephone, telegraph, and other quick means of transmitting messages. (5) Municipal supplies, embracing, among others, water, light, heat and power. (6) Waste disposal, including sewage, garbage, ashes and miscellaneous refuse. (7) Health, fire and police services. (8) Education in all its phases, not omitting art, natural history and other forms of instruction for both old and young. (9) Recreation, physical and mental. (10) Charities and miscellaneous.

Some of the topics named under these ten heads, and a number of others necessarily omitted in a mere outline, involve services rendered chiefly or wholly by private companies, but they need careful consideration from several points of view, and particularly from their relation to the municipal

revenues. From the latter viewpoint, also, attention should be given to possible revenues, even though only partially balancing expenses, from the disposal of city wastes. Another financial topic needing consideration, and which is closely related to starving one department in order to fatten another, is proper compensation to one department for services rendered to another. The most common and flagrant injustice of this sort is the practice of making the water departments of so many cities supply water to all the other city departments without a cent of credit, in some instances turning some of the water revenues into the general treasury in addition.

The foregoing is designed to be only an outline of the need and scope of a municipal program. In conclusion, one drawback and one particularly notable essential to such a plan may be mentioned. The drawback is the rapidly changing personnel of American municipal government, but adopt or make a good start on a municipal program and the changes in personnel might decrease, or it is barely possible, become less significant. The essential, mentioned just above, is a well-equipped bureau of accounts and publicity, to form a clearing house between the various municipal departments, to keep the whole city government in touch with the people, and to exchange information with other municipalities.

LETTERS TO THE EDITOR.

Stop-Planks, and Not Flashboards Used on the Tabeaud Dam.—Correction.

Sir: On p. 28, Fig. 12 and text, of your issue of July 10, you call certain things "flashboards" which are "stop-plank." Some fine day there will be a disaster caused by some loose-eared or loose-mouthed individual putting in stop-plank, when some one else has recommended the use of flashboards, so that the subject is not one of slight importance. A flashboard means a board or plank to retain water, so held that when the water rises more than a fixed mark (generally some 18 ins. or 2 ft. over the top of the flashboard), the fastenings of the board will yield and the boards be swept away. A stop-plank is ordinarily held by masonry grooves which can not let go their hold on the stop-planks, and which last named, as a rule, are difficult or impossible of removal during the progress of a flood such as would automatically be prevented or made harmless by flashboards.

As a well-conducted newspaper, I suppose you do not even remotely want to aid in the continuance of or in bringing about such confusion or consequent damage or disaster; others are in need of having their attention called to this same matter—hence this note is sent you.

"2 Wall St."

New York, July 11, 1902.

A Graphical Method of Finding Bending Stresses in Eyebars.

Sir: In finding the secondary stress in an eyebar due to its own weight, it is necessary to use the formula

$$P = \frac{4{,}900{,}000\ h}{P_t + 23{,}333{,}000\ \frac{h^2}{l^2}}$$

where

P = Stress in lbs. per sq. in. due to bending.
P_t = Working tensile stress in lbs. per sq. in.
h = Depth of bar in inches.
l = Length of bar in inches.

The formula is based on a value E = 28,000,000 and a weight of metal of .28 lb. per cu. in.

On account of the amount of work required to solve this equation repeatedly, the following diagram was constructed for facilitating the solution. It may possibly be of use to someone else.

To use the diagram, find in diagram A the intersection of the given values of h and l. Read this on the diagonal lines and add to this value the working tensile stress Pt, which gives the denominator of the fraction. In diagram B find the intersection of this value with h and read the value of P on the diagonal line.

The maximum values of P for the various working stresses and lengths are plotted by dotted lines. These may be used directly, with little excess for any usual height of bar, thus giving a safe value for P with one reading.

Yours very truly,

Wallace E. Belcher.

American Bridge Co.,
East Berlin, Conn., June 19, 1902.

The Length of the Vara, Again.

Sir: Referring to the communications in your issues of June 12 and July 3, regarding the length of the vara, as the unit of measure in Texas, I would say that Mr. Bullock gave the correct value as used in this state. Coming here fresh from the North, like Mr. Bullock, I ran up against the vara unexpectedly almost the first thing. In looking up authorities for its value, I found several varas, among them, the Cuban, 33⅓ ins.; Mexican, 32.97 ins.; Portuguese, 1 span = 0.7214 ft. and 5 spans = 1 vara; Spanish, 2 sesmas = 1 pie = 0.9273 ft., and 3 pies = 1 vara. Finding so many different values I enquired of local authorities as to the value used in Texas, and found it to be 33⅓ ins. The following table may be useful to engineers having to do with Texas land measures:

1 vara = 33⅓ ins.; 36 varas = 100 ft.;
1,900.8 varas = 1 mile = 5,280 ft.;
5,645.4 square varas = 1 acre = 43,560 sq. ft.;
1 league and 1 labor = 5,000 varas square = 4,605.6 acres = 14,104 ft. sq.;
1 league = 5,000 varas square = 4,428.4 acres = 13,889 ft. sq.;
1 labor = 1,000 varas square = 177.1 acres = 2,778 ft. sq.;

$$\text{Feet} = \frac{\text{vara} \times 100}{36}$$

$$\text{Vara} = \frac{\text{feet} \times 36}{100}$$

Respectfully yours, P. A. McCarthy,
Chief Engineer, Eastern Texas R. R.

Kennard, Tex., July 8, 1902.

Sir: As my communication of June 12, "Length of a Vara—Ask Questions," received such a severe criticism from Mr. J. C. Trautwine, Jr., in your issue of July 3, I will ask you to publish the following letter, which is self-explanatory. I think this very controversy will emphasize the importance of "asking questions." Yours,

W. P. Bullock, C. E.

Dallas, Tex., July 10, 1902.

Dear Sir: Replying to your favor of July 7, you are advised that we have not the leisure to prepare a statement for Engineering News, but with regard to the "vara" as 33⅓ ins. and 1,900.8 varas to the English mile of 1,760 yards. You can use this statement in any article you may prepare on the matter. Very truly yours,

Charles Rogan, Commissioner.

General Land Office, State of Texas,
Austin, July 9, 1902.

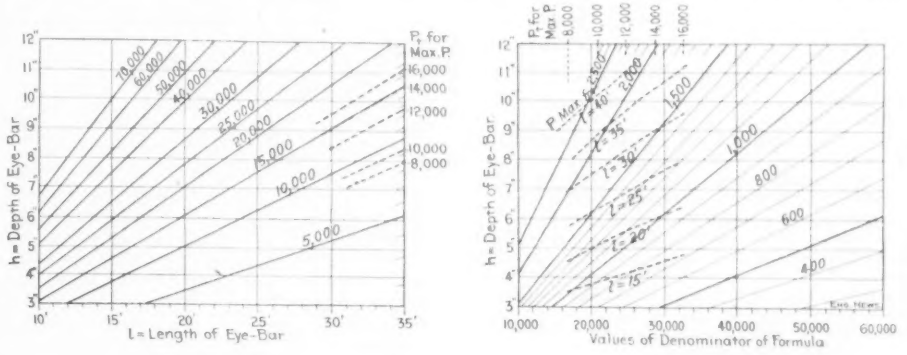


Diagram A. Diagram B.

A DIAGRAM FOR FINDING BENDING STRESSES IN EYEBARS DUE TO THEIR OWN WEIGHT: GRAPHICAL SOLUTION OF FORMULA

$$P = \frac{4{,}900{,}000\ h}{P_t + 23{,}333{,}000\ \frac{h^2}{l^2}}$$

Diagram A: Diagonal lines give values of $23{,}333{,}000\ \frac{h^2}{l^2}$.
Dotted lines give maximum P for any P_t.

Diagram B: Diagonal lines give values of P = stress in extreme fiber due to weight of eyebar;
Dotted lines give maximum P for any P_t and l.

A Criticism of the Grand Ave. Bascule Bridge at Milwaukee.

Sir: While in Milwaukee, Wis., last week, I took the opportunity of making a study of the Grand Ave. bascule bridge. It is certainly a good bridge, but some points of construction could have been better executed.

In the first place the bridge crossing the river at an angle instead of squarely makes the clear channel of 70 ft. awkward for big boats and only fairly available for those going up stream through the east channel of the Michigan St. draw. The structure should have been planned for a wider channel, the old abutments torn down and hence the bridge made more permanent though a little more expensive.

The foundations have not been properly designed: the total load on each concrete pier comes too near the front; there is a tendency to an uneven settlement, and of the main girders butting against one another at the center lock. I measured 3 ins. between the ends of the north girders and 4 ins. between the other two. This may be due to a fault in erection or the piers settling more on one side of the structure than on the other; at any rate, this is bad for the floor and the machinery.

Too much care cannot be taken in designing the trunnions and the main boxes on bridges of this type, for these are parts that cannot be easily replaced. The use of babbitt instead of bronze bushings in the main boxes of this bridge is poor judgment to say the least. The enormous load has already shifted the babbitt towards the caps, and the trunnions will soon touch the steel ribs at each end of the boxes. The customary oil grooves in the bushings must now be nearly flattened out and good oiling therefore is difficult if not impossible.

The heel locks work loose in their upper seats and are therefore useless. The writer fails to see of what great use they could be anyway. A block of oak would have been much cheaper, just as effective and less troublesome to the bridge tenders than the air buffer used.

Every time the bridge is raised the dirt on the swinging parts falls down on the machinery. This is bad, for a pebble or a spike can cause lots of trouble. By locating the break in the roadway in front of the trunnions instead of in the rear, this could have been prevented.

The attachment of the rack to the main girders is good, but the writer thinks it would have been better and cheaper to place it right on the center line of the main girders at the bottom flange.

The 7-ft. gear is not accessible for repairs.

The houses are too small and should have been built with a projecting platform in front, carrying the controller and the operating levers, thereby giving a chance to the bridge tenders to see further up or down stream and thus avoid delay in closing the bridge. This is very important, for Grand Ave. is the most frequented thoroughfare in Milwaukee. If a boat is going down stream the bridge-tender, who is on the west side, cannot see when the boat has cleared the structure and must wait until he is signaled from the east side or from the boat before he closes the west bascule. Hence delay in street traffic.

The wells in the piers should be kept empty to insure a better control of each leaf when in motion. If not, ice would then interfere with some gears.

As a whole it is certainly a remarkable structure.

Yours truly,

R. G. Vallée, M. E.

Chicago, Ill., June 21, 1902.

On the Loss of Head in Passing Water Through 2½-in. Fire Hose.

Sir: Mr. S. A. Charles' experiments on the loss of head in passing water through 2½-in. fire hose, published in Engineering News for July 3, seem so valuable that I trust he will pardon me for putting them into a more available form than the ultra-fashionable diagram, which he has chosen for their publication.

These experiments were made with standard 2½-in. rubber-lined fire hose, delivering water through a 1-in. nozzle, the pressure at the hydrant being steadily maintained at 100 lbs. per sq. in., and that at the base of the discharge pipe being measured by a gage, while the stream was actually flowing. Under these circumstances the pressure at the discharge pipe diminishes proportionately to the square root of the length of the hose, and therefor we have

$$P' = C\sqrt{L},$$

where P' is the loss of pressure per square inch in avoirdupois pounds; L the length of the hose in feet; and C a constant to be determined from the experiments.

Mr. Charles' experiments, and their reduction, are exhibited in the following table, which is self-explanatory:

L., ft.	$\sqrt{L}$.	P' obs'd, lbs.	$\frac{P'}{\sqrt{L}}$.	P' computed, lbs.	v, lbs.	v v.
50	7.071	13.0	1.838	15.2	+ 2.2	4.84
100	10.000	20.0	2.000	21.5	+ 1.5	2.25
200	14.142	30.8	2.178	30.3	— 0.5	0.25
300	17.321	38.8	2.241	37.2	— 1.6	2.56
400	20.000	44.3	2.215	43.0	— 1.3	1.69
500	22.361	50.0	2.236	48.1	— 1.9	3.61
600	24.495	54.5	2.225	52.7	— 1.8	3.24
700	26.458	58.5	2.289	57.0	— 1.5	2.25
800	28.284	61.9	2.188	60.8	— 1.1	1.21
900	30.000	65.2	2.173	64.5	— 0.7	0.49
1,000	31.623	70.0	2.214	67.9	— 2.1	4.41
1,549	38.730	77.6	2.004	83.2	+ 5.6	31.36
			25.801			58.16

The column headed $\frac{P'}{\sqrt{L}}$ contains the value of C given by each experiment, and from the mean of all we find

$$P' = 2.15 \sqrt{L} \text{ lbs.}$$

From that formula we compute the value of P′ for each experiment, and obtain the numbers given in the fifth column. The differences between these computed values of P′ and the observed values are the v's, contained in the column so headed; and from their squares we compute the probable error of P′, and find it to be ± 0.448 lbs. The complete expression for the result of these experiments is therefore

$$P' = 2.15 \sqrt{L} \pm 0.45 \text{ lbs.}$$

or, if P is the pressure per square inch, at the end of a line of hose L ft. long, having an initial pressure of 100 lbs. per sq. in., and delivering water through a 1-in. nozzle, then

$$P = 100 - 2.15 \sqrt{L} \pm 0.45 \text{ lbs.}$$

This formula seems to indicate that an initial pressure of 100 lbs. per sq. in. would be completely used up by friction in a line of hose about 2,200 ft. long. But Mr. Charles states that 3,000 ft. were once used in Lexington, Ky. It would be interesting to know the initial pressure on that occasion.

Very respectfully,

Wm. Harkness.

90 Mercer St., Jersey City, N. J., July 4, 1902.

The Intercontinental Railway.

Sir: I wish to protest against the general pessimistic tone of your editorials during the past few years on great engineering enterprises that are projected outside of New York or its vicinity. As an example, your contemptuous reference in your issue of June 19 to the proposed appropriation of $20,000 for investigating the Pan-American Ry. seems to me to be entirely out of place in the leading engineering journal of America. It probably would be unreasonable to expect such a project to pay any direct dividends on the investment for some years. But the Nation undertakes many legitimate enterprises that can not be expected to pay dividends. On the dividend basis a committee of bankers would probably condemn the Panama Canal or would have condemned a railway to the Pacific Coast, or the hundred odd millions invested in the Philippines, or the three hundred million invested in a navy.

Let us consider what we might expect for the country from the Pan-American Railway besides dividends. We have water communication with South America in common with European nations. With us alone is railway communication possible. South America is nearly twice as large as the United States. It has enormous mineral resources only partially developed. In the valley of the Amazon there is an agricultural region as large as the whole of the United States, with a soil equal to that of Iowa and the climate a perpetual summer, that is practically unsettled. Europeans are well adapted to an outdoor life in this region and with the exception of limited districts in the neighborhood of the falls find the climate very healthful. The testimony of competent witnesses who have actually spend considerable time in this region is unanimous on this point. With present railway speeds this region might be reached in about five days from New York. With the development of long-distance electrical transmission, speeds of 100 miles per hour and better may be reasonably expected before the middle of the present century. With this speed the distance from New York to the Amazon would be about 2½ days. To Rio de Janeiro or Buenos Ayres, which would include about nine-tenths of South America, would be about 3½ days. Nearly the whole country is tropical. Many agricultural products now unknown even by name to the most of us would become common necessities. Entirely new lines of manufacturing industries would be developed. The demand for our manufactures and for many of our agricultural products would be largely increased. Possibly there might be an occassional opening for a civil engineer. Last, but not least, it should be mentioned that with railway intercommunication the violation of the Monroe Doctrine becomes practically impossible. Yours,

J. A. Lewis.

718 10th St., N. W., Washington, D. C., June 28, 1902.

(Our correspondent's point of view is evidently so widely different from our own that we cannot hope to reach agreement. However, since others besides him may have misinterpreted our recent brief reference to the Intercontinental railway scheme, it seems worth while to explain our position more clearly, even at the risk of repeating the arguments against the project which we gave some years ago, when it was a subject of considerable public discussion.

The trouble with the Intercontinental railway scheme is that both freight and passengers can be transported between North and South America far cheaper and better on the ocean than by any railway line. Engineering News, as many of its readers will recall, has often taken strong ground as to the ability of railways carrying heavy traffic to compete with inland water routes by river and canal; but we have never claimed that the railway could compete with the ocean as a cheap highway for the movement of freight.

Our correspondent argues that an Intercontinental railway would be justified by its indirect benefits, even if it did not pay dividends on its cost; but it could not be a benefit unless it carried traffic, and it could not carry traffic between the United States and South or Central America in competition with ocean steamer lines.

Our correspondent refers to long distance passenger traffic as if that were justification for such a road; but he and every engineer in any way interested in railway construction ought to understand the important fact so ably set forth by the late A. M. Wellington, in his "Economic Theory of Railway Location," that long distance passenger traffic by itself never pays. The crude idea of most railway promoters of the amateur or the crank order, is that the best railway location is the straight line from one terminal city to another. Mr. Wellington showed that the best paying road is the one that touches the largest possible number of populous cities between its terminals, even if considerable detours from the direct line are necessary to reach them. Analyze the revenues of every railway line and the receipts from through passenger traffic will be found to be an exceedingly small proportion of the whole. In fact, we may go even farther and say that on a large proportion of the mileage of railways in the United States, the whole passenger traffic does not pay the cost of handling it.

If these things are true in this wealthy and populous land, what would be the situation of a railway traversing for a thousand miles the wild and almost uninhabited wastes of Central America.

Our correspondent thinks that nations ought to undertake enterprises even if they do not pay; but we cannot agree with him. If a railway or other enterprise will not warrant the investment of private capital, the nation ought not to compel its taxpayers to do what men of wealth are unwilling to do. The arguments in favor of a contrary course are plausible; but the financial ruin which has followed a large proportion of the railway subsidizing operations ought to be sufficient to prove such arguments to be false.

No one doubts that great development of Central and South America is among the probabilities of the future, and nothing can aid that development more than railway construction; but it will be local lines of railway, such as have been and are being built there, and not through lines built for political purposes, which will effect this development.

Finally, we deem it proper to express our conviction that our correspondent is wholly mistaken in his idea that Engineering News is unduly prejudiced in favor of New York enterprises. It may not be without interest to state that not one of those at present in charge of its editorial conduct or business management was born or bred in New York city or even in New York state.

As we had occasion to state not long ago, Engineering News is a New York publication only in the sense that its principal business office is located in New York city. Its aim is—we believe with some success—to cover the whole field of American engineering without regard to geographical limitations.—Ed.)

NOTES OF EXPERIENCE IN MASONRY CONSTRUCTION.

By F. A. Mahan, Major Corps of Engineers, U S. A.*

Having seen only recently several articles on the construction of different kinds of masonry, which appeared in Engineering News during the early part of the year, in connection with the failure of masonry dams in different parts of the South, I hope that I may be allowed, late though it be, to contribute my mite to the discussion.

As assistant engineer, or as chief engineer of some important construction works in masonry of all kinds, where it was my good fortune to be allowed always to work by day's labor and not by contract, a certain amount of experience has fallen to my lot.

The result of my observation of masons is this. As a rule they are ruined by much of the contract work on which they are employed. They are so hurried and harried by the contractors to run the work to an end as soon as possible, that at last, whatever they may have been at first, they become utterly careless and no longer wish to do good work. A certain amount of moral force is required to correct bad habits, and this moral force is rarely a possession of the man who works by the day. At the Davis Island dam, on the Ohio River, near Pittsburg, there were engaged three as good master-masons as any one could desire—an American, an Irishman and a Scotchman. What they found hardest to do was to prevent their men from slighting their work. Very few masons stayed a season through; they were discharged because they would not do their work properly. At the Buffalo breakwater there was much of the same trouble. Although the masons on that work were receiving $3.75 a day, as against $3.00 a day in the city, there was great difficulty in obtaining men. Those who did come rarely stayed. For a long time the cause of the trouble could not be determined. Finally the master-mason informed the writer that the trouble lay in the quality of the work the masons had to do. They preferred the smaller pay in town where they could slight their work to the full content of the contractors who employed them. They had lost the habit of good work, if they ever had it; and taking pains was too hard work for them.

The lock walls at Davis Island were made of large cut stone with very irregular rubble backing. The cut stone was regularly coursed, the courses averaging 20 ins. in thickness—if memory serves me rightly in the matter of figures dating back more than 20 years. The backing of the land-wall was of comparatively small rubble. I doubt whether any stone in it weighed as much as 1,000 lbs. The river wall had two-cut stone faces. The main part of the backing was built of the largest stones to be had from the quarries. Many of them were more than two courses high and weighed as much as two or three tons. When a block came out from the quarry too irregular in shape for economical cutting, it was used as backing without breaking it up. This could be done readily, as the traveling derrick used had an easy 5 tons capacity. So far as possible these big pieces were laid so as to back the entire thickness of one course of cut stone and half the thickness or more of the courses next above and below. When one of these stones was to be laid it was lifted clear of the ground by a derrick. While suspended it was washed perfectly clean by means of a heavy stream of water, from a powerful steam pump, assisted by hoes and brooms. During the washing process the mortar bed to receive the stone was laid. It was often 6 ins. deep. The mortar was brought in wheelbarrows and spread roughly with a shovel. The cleaned lump of stone, frequently of too irregular shape to be called a block, which weighed from 1,000 to 6,000 lbs., was lowered with the best side down on the bed of mortar, which was crushed out from underneath on all sides. Small well-washed stones were then rammed into the mortar so as to hold the big stone perfectly steady.

After four or five large stones had been set in this way, the voids between them were filled by means of wheelbarrow loads of mortar dumped in one on top of the other. Into these spaces were rammed stones of sizes to suit the voids. As each stone went down it left a hole in the mortar above it. This hole was filled up with more mortar before the next stone was put in. The operation was continued until it was impossible to ram in any more stones. No very skilled labor was required, as the use of a trowel was rarely necessary. The squeezed-out mortar was picked up with a shovel. The wall was tight. It was (trusting to memory again) 20 ft. high above the concrete foundation, 16 ft. wide at the bottom, and 7.5 to 11 ft. wide on top. The river rose on one occasion to within a couple of inches of the top of the wall, the lock inside being empty. Scarcely a drop of water appeared on the entire length of the wall, more than 600 ft.

*51 Avenue Montaigne, Paris, France.

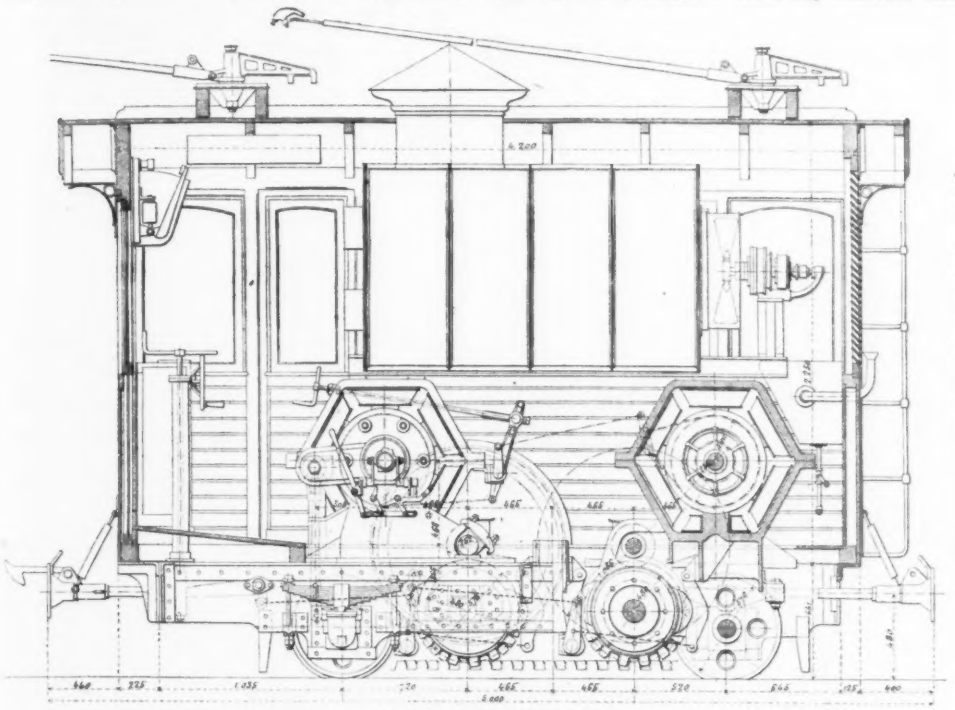
Naturally, the mortar was forced by the above operation into every irregularity in the surface of the stones. It is not seen how any voids or air spaces could have existed. The smaller stones were not driven down from the top alone, but in at the sides, and in any way that they could be forced into the mass of mortar. Each stone wedged the mortar more tightly around the preceding stones. No grout could have obtained an equally good result.

Grout was almost never used on any work with which I had to do. When it was used it was only when no way of avoiding it could be found. In order to flow properly the proportion of water must be so large that in hardening there must be an important reduction of volume. Furthermore, grout is run, as a rule, under a very small hydrostatic head which is not sufficient to force it into place. In carefully cut work, where it can be forced or led into place by means of proper tools, it may work well sometimes. If the stones be of irregular shape grout should never be used, unless it can be run under a head of 25 to 30 ft., and even then it were better to try to adopt some other way of building.

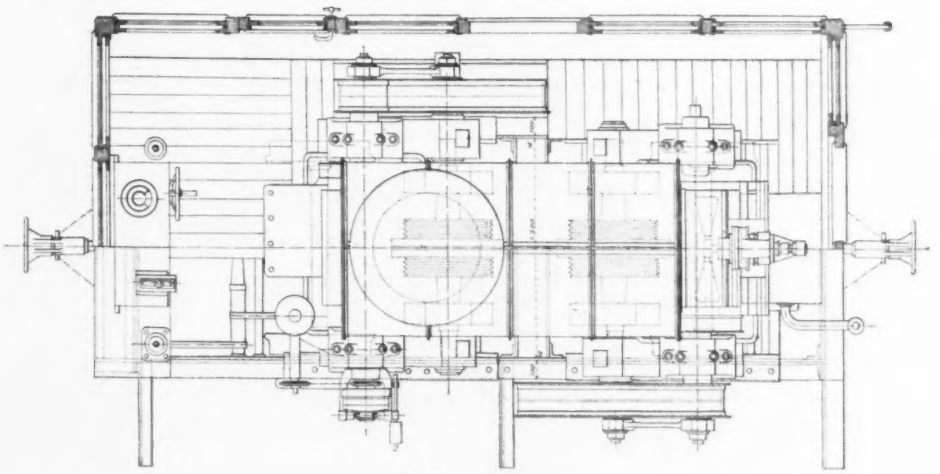
In the matter of concrete, after having laid some 300,000 to 350,000 cu. yds. in structures of various sorts, I think that too much stress is laid on the use of the minimum amount of water. Theoretically the minimum amount of water doctrine is slightly better, but I doubt greatly whether the quality of the concrete is so much improved as to justify the greatly increased cost of ramming required to produce the result. I have put it in so dry that it required twelve men's work during fifteen minutes to ram it, and so wet that one could not walk on it, much less tamp it. Of the two extremes, I prefer the latter for all practical purposes. I recall one occasion, during the reconstruction of the Buffalo breakwater, when a batch of concrete was sent out from the mixer so wet that it "slopped" around in the car which carried it to place. I had it thrown out to one side. Twenty-four hours later it had become so hard that a heavy blow of a stone pick produced no effect on it. I used all my concrete sufficiently moist to allow of ramming all the coarse material below the surface. With dry concrete this cannot be done. It is nearly impossible to avoid nests of broken stone which will not ram in. The stones can be driven into the mass and the mortar forced around them much better when the mixture is not so very dry. I believe that concrete made of the consistency of good stiff mortar gives the best results. When too wet, voids are formed by the shrinkage of the fluid mortar; when too dry, the voids cannot be filled because the mortar is not sufficiently plastic. In any case the mortar should be slightly in excess of the exact quantity required to fill the voids in the coarser materials.

So far as the use of blocks of stone in concrete is concerned, I doubt its utility or its cheapness. I have tried the use of such blocks and I have invariably given it up except in fortification work, where there is a special reason for its continuance. Such blocks are always irregular in shape, and I have nearly always found it necessary to bed each one of them in mortar to obtain good results. If this be not done the block is apt to rock on its bed, to be displaced by ramming the concrete, and to have voids underneath it. It is generally not practicable to set such stone while the concrete is in condition to receive them. The attempt to do this causes the men who are setting the stone to interfere with those working on the concrete. If a little time be left between the operations of the two gangs, the surface of the concrete is frequently broken up by the stone gang. If the concrete be allowed to become perfectly firm before the stone blocks are laid, there is no chance of a bond being had between the stone and the concrete, so the bed of mortar must be used. A gang of twelve men will ram well and easily a cubic yard of concrete in six minutes. It is what I always expected in straight ordinary work. This gives an area of six square yards 6 ins. thick. The stone gang can rarely cover this area properly in the same length of time, for the reason that it is the rare exception when more than four men can work on this job. If the stone have to be laid at a certain height above ground one of two things must happen: either there must be a special derrick to lift the stone, or the concrete work must stop while the concrete derrick lifts it.

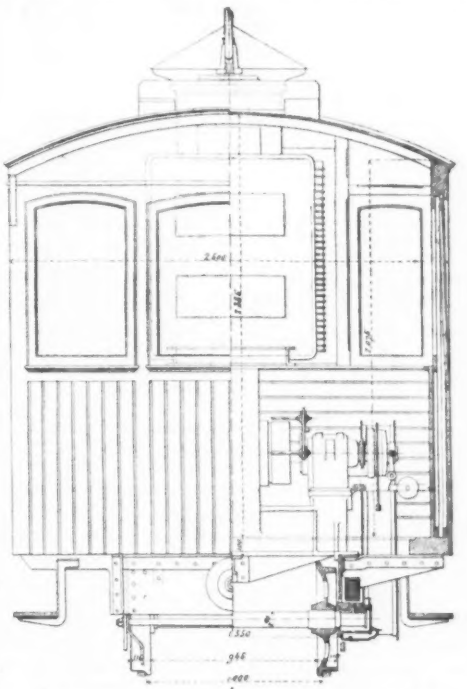
In the construction of reservoir dams I can see but two really satisfactory forms of masonry. One, according to the mode of building the backing of the river wall at the lock of Davis Island; the other the system of comparatively small stone, one-to-two-man stones, of irregular shape, forced in among each other, with plenty of good cement mortar, as used, for example, in the Goufre d'Enfer and the Pas de Riot dams near Saint Etienne, France. Of the two the latter is probably a little the better method.



Longitudinal Section.



Sectional Plan.



½ End Elevation. ½ Transverse Section.

FIG. 1. ELECTRIC LOCOMOTIVE FOR THE RACK-RAILWAY BETWEEN BEX AND VILLARS, SWITZERLAND.

Prof. A. Palaz, Consulting Engineer; Victor Ryncki, Chief Engineer.

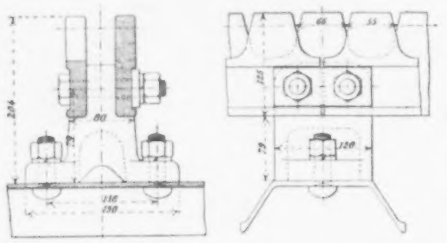
THE ELECTRIC RACK-RAILWAY OF BEX-GRYON-VILLARS, IN SWITZERLAND.

This new line, as described in "Le Genie Civil," is divided into three sections, two operated by adhesion and a central portion fitted with a rack-rail. The first section starts at the station of the Jura-Simplon Railway, at Bex, and extends to Salines du Bevieux, a distance of two miles. It was opened to traffic on Sept. 10, 1898, and follows the main highway between these two points.

The second, or steep grade section, is built upon its own roadbed, and is provided with an Abt rack-rail. Commencing with a 4% grade, the line

crosses the Avancon River by a metal bridge of 111.5-ft. span, and at Sublin it reaches the heaviest grade, one of 20%. Near Sublin the electric generating plant is located, utilizing for power the fall in the River Avancon. After decreasing to a 4% grade in the little valley of Fontannaz-Sular, the line again ascends rapidly, and to overcome the grades a loop is made, including a tunnel 590 ft. long, built on a 20% grade, and on a curve with a radius of 262.4 ft. At the village of Gryon, the rack-rail ends, and adhesion only is used from that point to the terminus at Villars, 2.7 miles beyond Gryon. The total length of the line is about 7.5 miles; and in this distance the line rises 2,558 ft.

On the upper section of the line the only important structure is the bridge across the Gryonne. This bridge is a cantilever structure, the



Cross Section. Side Elevation.

Fig. 2. Details of Rack-Rail.

first of its type built in Switzerland. It has a total length of 466 ft., and is placed 138 ft. above the river.

In the portions of the railway operated by adhesion the grades were limited to 6%. The radius of curves on the rack-section is generally 262.4 ft.; though at the station the radius is sometimes as low as 196.8 ft.

The cars are operated by a continuous current of 600 volts by an overhead wire, from which current is taken by sliding shoes on trolley poles; one on each end of the car. The power station is located near the middle of the heaviest grade, which greatly simplifies the distribution of current.

Both motor cars and electric locomotives are used on the road, but only the latter are fitted with pinions for service on the rack section, and the motor cars are hauled as trailers, therefore, on this section.

The electric locomotive is shown in the accompanying drawings, and will be seen at once to be of quite unusual construction. The motors are connected to the rack-pinions by intermediate gearing, effecting a total speed reduction of 8 to 1. Thus, with the motor making 525 revolutions per minute, and a rack-rail pinion with 23 ins. diameter pitch circle, the speed is but five miles per hour. The motors are of over 100 HP. each, and the locomotive can haul a load of 15 tons up the 20% grades of the line at a speed of five miles per hour.

Naturally great care is requisite in the braking equipment of a line on such steep grades. Under all normal conditions the descent of the rack section is made with the motors reversed and acting as dynamos. The current which they generate is turned into a large rheostat built up of corrugated iron and placed in the center of the car, directly over the motors. To prevent this rheostat from overheating an electric fan is installed to blow air through it, and the current connections are so arranged that the fan is automatically set in operation whenever the locomotive is descending, but it is switched off when it stops or runs upgrade.

In service this method of braking has been found to give most excellent results, and to permit very close regulation of the speed. For emergency use, however, the locomotive is also equipped with two hand-brakes acting upon the rack-pinions; a hand belt-brake for the motors, and a screw-brake acting automatically upon the motors when the normal speed of 7.5 miles per hour is exceeding in descending, or when the electric current is intercepted from any cause whatever.

Besides these brakes on the locomotive, the motor cars which it hauls as a trailer has brake shoes on every wheel of the car and a brake acting on the rack-rail besides.

The total cost of the railway and rolling stock was about $282,000, divided as follows:

Cost of first section	$60,000
Cost of second section	176,000
Cost of third section	46,000

The work was executed on the plans of Prof. A. Palaz, under the direction of Victor Ryncki, as chief engineer, and Mr. Wohnlich, engineer in charge.

BOOK REVIEWS.

WORLD'S CABLE DIRECTORY.—International Cable Directory Co., 17 State St., New York; Salisbury House, London Wall, E. C., London. Cloth; 8 × 10½ ins.; pp. 438.

This cable directory of the world is issued in conjunction with the Western Union telegraphic code system, which is now accessible to correspondents in almost all countries. The alphabetical list of names, with address, business and cable address, is arranged by continents, countries and cities. A second directory classifies the names under business headings, with the business stated in English, German, French and Spanish, and a general index by towns and cities refers to the pages containing detailed information.

THE STUDY AND DIFFICULTIES OF MATHEMATICS. —By Augustus De Morgan. Cloth; 5½ × 8 ins.; pp. 288; 13 illustrations. Edited by Thos. J. McCormack. Second Reprint Edition. Chicago: Open Court Publishing Co. Price, $1.25.

This book seems fully as appropriate and valuable as when it was first published, in 1831, by the Society for Diffusion of Knowledge (England). It is a logical exposition of the science of mathematics, beginning with an analysis of the concepts fundamental to arithmetic, and building up on these, in a strictly logical course of reasoning, through arithmetic, algebra, geometry and trigonometry. It outlines the fields of these sciences and makes clear the nature of the thought processes involved in their development and use. The language is clear and the reasoning keen and luminous.

METHODS OF GAS ANALYSIS.—By Dr. Walther Hempel, Professor of Chemistry in the Dresden Technische Hochschule. Translated from the Third German Edition and Considerably Enlarged by L. M. Dennis, Professor of Analytical and Inorganic Chemistry in Cornell University. New York: The Macmillan Co. Cloth; 6 × 8 ins.; pp. 490; tables and 136 figures in the text. $2.25.

This comprehensive treatise on gas analysis, the first edition of which appeared a number of years ago, has been improved by the addition of considerable new matter. The portions most likely to interest engineers deal with the collection of gases from furnaces, mines and gasholders, taking samples of air and securing absorbed gases from liquids; the chapters on the analysis of illuminating gases, and on gases produced by bacteria; and a chapter each on colorimetric determinations of the heating power of fuel and of gases.

RIDER'S LITTLE ENGINEER.—A Pocket Book of Engineering and Other Data Relative to Many Subjects. Instant Answers for Engineers, Contractors, etc. By Joseph B. Rider, C. E. South Norwalk, Conn.; Joseph B. Rider. Flexible leather, with flap, 4¼ × 7 ins.; pp. 480; illustrations and tables. $3.

Besides a variety of elementary rules and numerous tables of equivalents, weights, measures and money values, this handy little pocket book contains much information for civil and municipal engineers. Portions of the book partake of the nature of a cyclopedia, as well as an engineer's pocket book. There are notes and tables of cost of executed work, particularly of cast-iron water mains of various sizes. The book contains a goodly number of original tables, compiled, it is presumed, because the author could not find similar or so convenient ones elsewhere. The book contains many typographical errors, not all of which have been noted in the list of errata. It is to be hoped that the proof-reading of the tables was better than of the text.

WATER SUPPLY.—(Considered Principally from a Sanitary Standpoint.) By Wm P. Mason, Professor of Chemistry, Rensselaer Polytechnic Institute. Third Edition, Rewritten. New York: John Wiley & Sons. Cloth, 6 × 9 ins.; pp. 448; tables and illustrations, 84.

The author has here omitted the chapters on the chemical and bacteriological examination of water, which occupied 90 pages of the first edition, and are now published separately. Some other matter has also been omitted, but enough has been added to keep the size of the volume within 56 pages of the original. The new matter seems to be well distributed and to bring many of the remaining chapters up to date. There are, however, a number of tables of annual statistics which have not been revised, but some of these were never very essential to

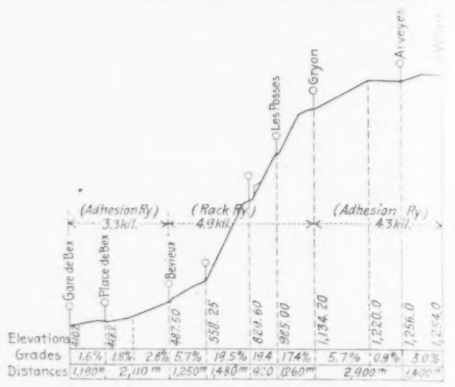


Fig. 4. Profile of Bex-Villars Railway.

volume devoted chiefly to the sanitary phases of water supply. In fact, the book would not suffer if still further chapters and sections were omitted, and prospective purchasers would doubtless welcome a second reduction in price.

SCHOOL ARCHITECTURE.—A General Treatise for the Use of Architects and Others. By Edmund March Wheelwright, F. A. I. A. Boston: Rogers & Manson. Cloth, 8 × 10½ ins.; pp. 324; 168 illustrations. $5.

This book is a comprehensive review of the best practice in school architecture throughout those countries where educational facilities have been most liberally and intelligently provided. Its author was for many years city architect of Boston, and in that capacity designed a number of school buildings and did other notable work in public architecture. The many illustrations in the book, comprising scores of floor plans and numerous interior and exterior views, are particularly valuable. The text supplements, but does not duplicate, the information conveyed by the floor plans.

There are separate chapters on manual training schools and on "Heating, Ventilation and Sanitation." The book closes with "Specifications for an American School." If every school board and every school architect would study and profit by this book, the next five or ten years would yield a remarkable improvement in school architecture.

VICTORIAN MUNICIPAL DIRECTORY.—With Gazeteer; also Water-Works and Irrigation Trusts Record for 1901. Melbourne, Australia: Arnall & Jackson. Half leather; 5 × 8 ins.; pp. 647. $3.60.

Besides a very full list of local officials this book gives a variety of descriptive information regarding all the municipalities and many outlying villages of Victoria. There are also particulars regarding 107 water-works and irrigation trusts, a directory of municipal, land, water supply and mining surveyors; an official directory of, and other information relating to, the commonwealth of Australia and the State of Victoria. A number of pages are devoted to the Melbourne and Metropolitan Board of Works, of which Mr. Wm. Thwaites, M. Inst. C. E., is engineer-in-chief. This board consists of a chairman and 39 members elected by the councils of the 24 cities, towns, boroughs and shires composing the metropolitan district. The board has charge of the water supply, sewerage and house drainage of its district. It is interesting to note

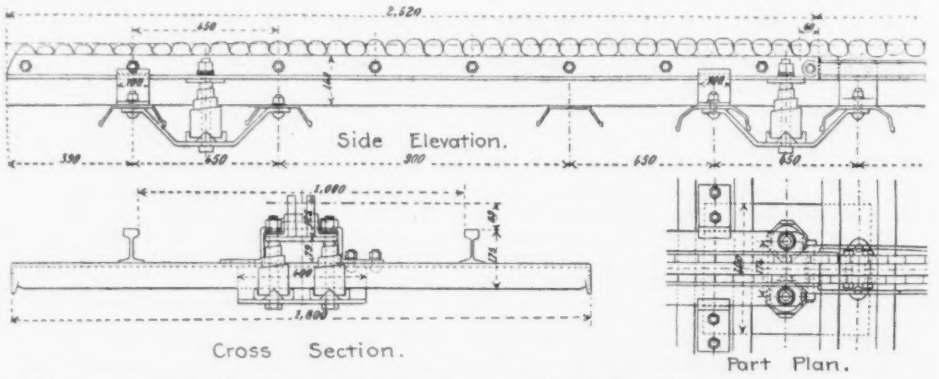


FIG. 3. SECTION OF RACK-RAIL AT END OF RACK, SHOWING SPRING SUPPORTS.

that 9 of the 11 cities and 7 of 10 towns in Victoria are within the jurisdiction of this board. The street railways of Melbourne and vicinity were constructed by the Melbourne Tramways Trust and are leased to the Melbourne Tramways & Omnibus Co., the lease extending to July 1, 1936.

VERE BAUWEISEN UND BAUWERKE AUS BETON UND EISEN. II. Theil.—Eine Belastungsprobe mit Decken nach System Hennebique. Von Fritz von Emperger. Vienna: Lehmann & Wentzel. Paper; 10 × 13 ins.; pp. 18.

The first section of this work appeared over six months ago; the third and fourth are promised for the present summer. Part II. contains a summary of results of a test made toward the end of 1899 on two ceilings of concrete-steel, and details the method of calculation used by Mr. J. E. Brik in analyzing these results, as published in the "Allgemeine Bauzeitung." It then reprints a series of letters, originally published in the Austrian "Wochenschrift für den öffentlichen Baudienst," in which Mr. Brik and Mr. von Emperger carry on a heated discussion relative to the above-mentioned method of calculation. The purpose of the author in presenting this matter as part of a work on concrete-steel construction is, as we are informed in a private communication, to show "how under a highly scientific disguise very serious errors may come before the public," and "that the new field of concrete-steel demands closer study." It may be expected that when the work is completed by the issue of the remaining two parts, it will offer a large amount of valuable information relative to the application of concrete-steel and to the experience that engineers have had up to the present with this material.

METALLURGY OF CAST IRON.—A Complete Exposition of the Processes Involved in its Treatment, Chemically and Physically from the Blast Furnace Through the Foundry to the Testing Machine. By Thos. D. West, M. Am. Soc. M. E., Cleveland, O. The Cleveland Printing & Publishing Co. Cloth, 4¾ × 7½ ins., pp. 627; illustrated. $3.

This is the third and latest edition of this important book, and in it the matter of the first and second editions has been extensively revised and added to. The author in his preface says:

Part of this new matter constitues 20 new chapters embodying researches, experiences, experiments, discoveries, and illustrations that have been secured by the author since the publication of the first edition in 1897. To provide space for this large addition of new matter, 13 chapters treating of cupola practice, published in the first two editions, have been transferred to the "Moulders Text-Book," leaving this work to the treatment of subjects more appropriate in its title.

The book is divided into three "parts" containing, respectively 27, 20 and 91 chapters. The first 27 chapters treat of manufacture and use of coke, properties in ores, operations of blast furnaces, the different brands of pig iron, and of how to purchase and use them intelligently. Part two treats of the elements in cast iron and their physical effects, utility of chemical analyses, and how to use them in making the different castings. The last 230 pages, comprising Part III., discuss the properties of and methods of testing molten iron, disclose phenomena in the actions of cooling metal and present results of tests in all kinds of irons and best methods for testing. In this part of the book the author calls attention especially to Chapter 70 as being valuable for presenting a compilation of the American Foundrymen's Association's tests, in which each grade was poured from the same ladle of iron within about 20 seconds, thus causing all the bars of each grade to be poured at the same temperature, something that had never been achieved previously in obtaining records of test bars.

SEWAGE AND THE BACTERIAL PURIFICATION OF SEWAGE.—By Samuel Rideal, D. S. C. (London), Author of "Water and Its Purification" and "Disinfection and Disinfectants." Second Edition. London: The Sanitary Publishing Co. New York: John Wiley & Sons. Cloth, 6 × 9 ins.; pp. 308; tables, 9 plates and 40 figures in the text. $3.50.

Both the rapid progress in sewage purification and the demand for information on the subject are illustrated by the appearance of a second edition of this book within about a year of the first edition. For England, with which the book is chiefly concerned, the author appears to have worked in the latest information available up to the beginning of 1901. Most of what is said about American experience is based on the special reports of the Massachusetts State Board of Health for 1890, and the latest citations from those reports are from the one for 1899.

After condensation of parts of the original work and additions of new matter, the book shows a net gain of about 30 pages, fairly well distributed through the twelve chapters. The chapter subheads, save for six additions, are identical with those of the first edition. A few illustrations have been omitted and a few added, resulting in a net gain of two. The book was good before the revision, and is, of course, better now.

In the next edition it would be well to make clear that the 16 intermittent filter beds of one acre area each at Worcester, Mass., are not a part of the experimental plant of the Massachusetts State Board of Health. Just how many septic tanks there were in this country when the author wrote the words, "one of the few installations in the United States on the Exeter principle," etc., we cannot state, but in our issue of April 3, 1902, we published a list of 22 municipal plants in places of over 3,000 inhabitants. We note the absence of information regarding the Exeter septic tank specifically later than 1897. This is in line with the comment in our issue of May 29, to the effect that little or nothing regarding actual conditions at Exeter had been heard for several years.

MUNICIPAL ENGINEERING AND SANITATION.—By M. N. Baker, Associate Editor of "Engineering News." New York: The Macmillan Co. Leather back; 5 × 7 ins.; pp. 317. $1.25 net.

Since this book was written by a member of the editorial staff of this journal, we shall merely give an idea of its aims and scope. The preface states:

This volume is intended for that large and rapidly growing class of persons who, either as officials or as citizens, are striving to improve municipal conditions. It is designed to be a review of the whole field of municipal engineering and sanitation, rather than an exhaustive study of one or a few branches of the subject. The most vital points, however, under each class of activities and interests have been dwelt upon, the underlying principles stated, and, in many instances, details from actual practice given.

While it is not to be expected that engineers and sanitarians will find in the book much relating to their specialties that is new to them, it is believed that the number and variety of subjects treated, and the comparative newness of some of the topics, will make the book helpful even to professional men.

The book is in six parts, each containing a number of chapters. Under "Ways and Means of Communication," streets, pavements, sidewalks, subways, railway grade crossings, street railways, ferries, telephones and other subjects are discussed. "Municipal Supplies" includes water, ice, milk, markets, office buildings, and light, heat and power. The "Collection and Disposal of Wastes" covers sewerage and sewage disposal, street cleaning and allied topics, garbage disposal, cemeteries and crematories for the human dead. The "Protection of Life, Health and Property" includes fire protection, smoke abatement, public baths, parks and gymnasia, and various other topics. In the final section, on "Administration, Finance and Public Policy," such topics as city charters, municipal experts, the work of boards of health, municipal franchises and ownership, uniform accounting, taxation and assessments for benefits are discussed.

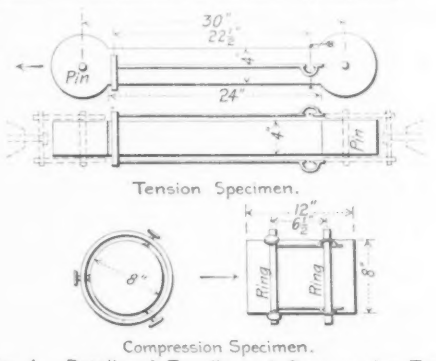


Fig. 1. Details of Tensile and Compressive Test Pieces for Testing Strength of Concrete.

The book is one of a series known as the "Citizen's Library of Economics, Politics and Sociology," edited by Dr. R. T. Ely, Director of the School of Economics and Political Science, University of Wisconsin. A number of chapters are credited in the preface to the author's wife, Ella Babbit Baker, including those on "Noise Suppression," "Municipal Bath and Wash Houses," and "The Sanitary Protection of the Milk Supply."

THE ENGINEERING INDEX.—Five Years, 1896-1900. Edited by Henry Harrison Suplee, M. Am. Soc. M. E., Mechanical Editor of "The Engineering Magazine," Assisted by J. H. Cuntz, C. E., M. E. New York: The Engineering Magazine. Cloth; 7 × 9½ ins.; pp. 1,030. $7.50.

Not so very long ago there was a dearth of information on engineering subjects, but that time has largely passed and the problem now is to ascertain quickly just what there is in print on a given subject. So far as books are concerned, this may be a comparatively easy task, but in these days of rapid progress a book is scarcely off the press before it is out of date. Anyone who wishes to keep posted must closely follow the engineering journals and society proceedings, and when any but the most current subjects are to be looked up nothing short of a consolidated index to periodical literature will enable a man to cover the field with dispatch. If each entry in a topical index be supplemented by a brief note indicating the nature and extent of the article the value of the index may be increased many fold. Just such a work is the volume before us, which covers nearly 200 American and foreign periodicals for the five years, 1896 to 1900, and includes a total of nearly 40,000 entries. The volume is the third one in a series, the first entries for which appeared in "The Journal of the Association of Engineering Societies" for January, 1884. The entries for the years 1884-91 were published by the association named under the title, "Descriptive Index of Current Engineering Literature." The work was begun and continued until the close of 1895 by and under the direction of Prof. J. B. Johnson, M. Am. Soc. C. E., late Dean of the College of Mechanics and Engineering of the University of Wisconsin. The second volume of the series for the years 1892-5 was published by "The Engineering Magazine," under an arrangement by which Professor Johnson's work ceased. The periodical named, as many of our readers know, publishes in its monthly issues an index to current engineering literature, on the same general, but much extended, plan as that followed for so many years in the "Journal of the Association of Engineering Societies." The present volume is a classified rearrangement of the monthly entries. The publishers promise a yearly consolidated index, to begin with the entries for 1901. The arrangement of the present volume is much better than that of the first one, and infinitely better than that of the second one. We take particular pleasure in mentioning this fact, inasmuch as we felt compelled in reviewing the second volume to condemn the slovenly manner in which the separate entries had been arranged. Considering the magnitude and difficulties of the task, the arrangement of the present volume is remarkably good. In most instances where entries under some other head would seem preferable cross-references under that head locate the articles in question. One exception to this is the entry under "Cost-Keeping" of three articles on "Depreciation," which do not appear in any shape under the heading last named. Of a different sort is the entry of one article under "Dam, Clinton, Mass.," and two other articles on the same structure under "Dam, Wachusett." The first entry relates to an article which appeared before the dam was officially named. We would suggest, in conclusion, that the "Engineering Index" having become so bulky that annual volumes are considered advisable, it will ere long be necessary to consolidate the three volumes for the period 1884 to 1900, and also the proposed annual volumes, but that when that is done perhaps the work should be divided by subjects into three or more grand divisions. At the same time the faulty and most exasperating arrangement of the first two volumes, particularly the second, could be remedied.

THE MUNICIPAL YEAR BOOK OF THE UNITED KINGDOM FOR 1902.—Edited by Robert Donald, Editor of "The Municipal Journal." London: Edward Lloyd. Cloth; 5 × 8 ins.; pp. 584. $1.40.

The valuable features of former issues of this annual are maintained in the present volume, which presents a wide range of information and a full official directory for the municipalities of Great Britain. The information on telephones, water-works, electric and gas lighting, street railways, housing the working classes and technical education is, for the most part, given separately. The sections on these subjects indicate the strong movement for municipal ownership and operation in England.

The first municipal telephone system in Great Britain was inaugurated in July, 1901, and in five months the municipality obtained "over six times as many subscribers as the company had obtained during its first eight years' work." In August, 1901, Glasgow began a municipal telephone service with about 5,000 subscribers. By the close of the year 45 municipal telephone systems had been approved or were under consideration.

This "Year Book" is practically limited to municipal undertakings, leaving water or gas works under private ownership, for instance, out of account. It contains few summaries, and those given are meager. Nevertheless, as an official directory and a reference book on municipal activities in England, Wales, Scotland and Ireland the volume is highly praiseworthy.

TESTS OF REINFORCED CONCRETE BEAMS.*

By W. Kendrick Hatt.†

GENERAL STATEMENT.—The writer has elsewhere‡ presented a theory of the strength of reinforced concrete beams. Since that time he has had an opportunity to carry out a series of tests on such beams in the Laboratory for Testing Materials of Purdue University, with a view to determining the reliability of the theory. The present paper describes the methods and records the results of the tests. Incidentally the theory advanced in the article cited has been improved and the results of its application compared with the observed facts of the test.

The writer desires to acknowledge the faithful services of Messrs. A. E. Kemmer, H. O. Garman, C. Klueh and M. R. Keefe, senior students in the Department of Civil Engineering, who, as a thesis exercise, under the writer's supervision, molded the test specimens, tested the same and worked up the data obtained.

The scarcity of recorded data giving results indicative of the nature of reinforced concrete under flexure is the motive which prompts the presentation of the results of the series of tests described below. The theoretical analysis is also believed to be of value.

TESTS.

These tests include observations of the load-deflection curve for beams of broken-stone concrete reinforced with 1% and 2% of wrought iron§ located at different positions in the cross section of the beam. A few tests on gravel and cinder concrete beams are added. The

*A paper read at the annual meeting of the American Society for Testing Materials.

†Professor of Applied Mechanics, Purdue University, Lafayette, Ind.

‡Proceedings Indiana Engineering Society, Jan., 1902; Eng. News Feb. 27, 1902.

§More exactly $^{693}/_{840}\%$ and $^{1392}/_{840}\%$.

beams were 8 ins. × 8 ins. in cross-section, and were tested on a span of 80 ins. under a center load. Beams Nos. 1 to 17, were 1–2–4 stone concrete; beams Nos. 18, 19 and 20 were of 1–2–4 cinder concrete; beam No. 21 was of 1–5 gravel concrete. The reinforcement was placed either 1 in. or 2 ins. from the bottom. Accompanying the tests on flexure is a determination of the following elements of the strength of the constituents: Moduli of elasticity and strength, in tension and compression, of the broken-stone concrete; the strength of the cement in tension and compression; the elastic limit and modulus of elasticity of the iron; analysis of the sand and stone; tests of the adhesion between the iron and the concrete.

CEMENT.—The cement used was Peninsular brand of Portland cement, made from marl and clay at Jackson, Mich. The strength of the cement in lbs. per sq. in. is shown by Table I:

TABLE I.—Showing Tensile and Compressive Strength of Portland Cement Used in Making Reinforced Concrete Beams.

Age	24 hrs.	7 days.	1 month.	3 mos.
Tensile Strength of Standard Briquettes.				
Neat	346	803	.....	802
3 : 1	78	412	505	626
Compressive Strength of 2-in. Cubes.				
Neat	2,402	7,337	9,537	9,740
3 : 1	610	3,145	5,860	8,005
Ratio of Compressive to Tensile Strength.				
Neat	6.94	9.13	.....	12.14
3 : 1	7.82	7.63	11.60	12.78

Slow setting; Fineness—98% through No. 100 sieve; boiling test—O. K.

BROKEN STONE.—The broken stone was limestone; the entire product of the crusher was used; 73% was retained on a ¼-in. sieve; all passed through a 1-in. sieve.

SAND.—The sand used was clean, sharp pit sand; 84% was retained on a No. 30 sieve; 20% was retained on a No. 20 sieve.

GRAVEL.—The gravel used was good quality of coarse gravel.

CINDERS.—The cinder used was that of bituminous coal, taken directly from the cinder pile, and contained about 50% fine ash.

MIXING, MOLDING AND STORAGE.—The cement and sand were thoroughly mixed in a dry condition; and then the water was applied; then the broken stone was added to the mortar and the whole thoroughly mixed. The concrete was fairly dry concrete intended to be plastic after a thorough ramming. About 5.5% of water by weight was added. Some of the specimens were more wet than others; these facts are noted in the individual cases in Table VI. The specimens were stored in dry sand and the rate of hardening was rather more rapid than that to be expected in the case of a cement walk. The specimens were taken from the molds as soon as they could be handled without danger of breakage. This period was from 24 to 48 hours in case of the beams made in the early spring, and from 12 to 24 hours in case of beams made at a later period.

TABLE II.—Showing Results of Tension and Compression Tests of Concrete.

Tension.

No.	Kind.	Age in days.	Modulus of elasticity, lbs. per sq. in.	Elongation at rupture, 1 part in	Strength, lbs. per sq. in.	Where broken.
	Stone.					
1.	1–2–4	35	2,700,000	11,660	300	At pin.
2.	1–2–4	33	2,400,000	8,750	305	"
3.	1–2–4	28	1,400,000	4,400	360	"
4.	1–2–4	26	1,900,000	7,700	280	"
Av'g of plain			2,100,000	7,000	311	....
5.	1–2–4	28	.....	910	281	Body.
	1% iron.					
6.	"	26	.....	1,310	313	"
Av'g of reinforced			.....	1,140	297	....
	Plain cinder.					
7.	1–2–4	11	.....	.....	82	Body.

Note.—Values for reinforced specimens are not corrected for stress in iron.

Compression.

No.	Kind.	Age.	Modulus of elasticity lbs. per sq. in.	Elongation at stress of lbs. per sq. in.	Compressive strength
	Stone.				
1.	1–2–4	9	4,702,000 at	750	2,880
		9	3,940,000 "	1,500	
2.	1–2–4	14	4,340,000 "	750	2,575
		14	3,680,000 "	1,500	
	Cinder.				
3.	1–2–4	9	558,000	...	495
4.	1–2–4	9	553,000	...	595
5.	1–2–4	7	630,000	...	416
	Gravel.				
6.	1–5	6	2,088,000	...	1,185

$n = \frac{E_c}{E_t}$ = 2.17 at 750 for A. } Stone concrete.

$m = \frac{E_c}{E_t}$ = 12.8 at 1,500 for crack. } Stone concrete.

SPECIMENS.—The compression tests were made on cylinders 8 ins. in diameter, and 12 ins. high. The tension tests were made on bars of square section 4 ins. on the side. The form of the tension specimens is shown in Fig. 1. These tension specimens were not well designed, since the heads pulled off before rupture occurred in the body of the bars. The tension tests included both plain bars and reinforced bars. Wooden molds were used for these specimens. The beams were 8 ins. square and 80 ins. between supports. The steel mold for these beams was built up of two channels and a bottom plate. The adhesion between the iron and concrete was determined by observing the load necessary to pull iron rods out of 6-in. cubes of concrete.

METHODS OF TESTING.

COMPRESSION.—Tests were made both on plain cylinders and on cylinders reinforced with rods parallel to the axis of the cylinders. The tests on these reinforced specimens are not quoted below. These reinforced cylinders were weaker than the plain cylinders, and the forms of rupture were abnormal. The shortening of all the cylinders was observed at three points around the circumference by means of hook micrometers attached to a low steel ring (Fig. 1) and having contact with an upper steel ring, the gage length being about 6½ ins. The contraction was noted by means of an electric current and the micrometers read to 1-1000-in. and estimated to 1-10,000-in. These tests were satisfactory.

TENSION TESTS.—The extensions of the tension bars were read on hook micrometers attached to an upper yoke and having contact with fine wires fixed by plaster of Paris to the specimen at the lower end of the gage length. The contact was made known by the passage of the electric current; the gage length was about 17½ ins.

These tension tests were not satisfactory since, as noted above, the heads of the bars, with some exceptions, pulled off. It is believed, however, by the writer that the strength of the body of the bars did not differ greatly from the loads recorded at the point of rupture of the head. The strength of the reinforced bars which broke in the body was nearly the same as that of the plain bars which broke in the head, viz., about 300 lbs. per sq. in. The stress-strain diagrams obtained in case of the reinforced bars are important elements of the tests.

FLEXURE.—The beams were simply supported at the ends and loaded with a center load. The deflections were read at the middle of the span on both sides of the beam from hook micrometers attached to the beam and making electric contact with wires stretched between points directly over the supporting knife edges, and 4 ins. above the latter.

In the three kinds of tests mentioned, loads were applied by increments, and the deformations noted. The loads were released at intervals in order to obtain the set.

RESULTS OF TESTS.

FLEXURE TESTS.—The load-deflection diagrams obtained from the concrete beams are shown in Figs. 2, 3 and 4. Notes on condition of concrete and on rupture of beams are given in Table VI.

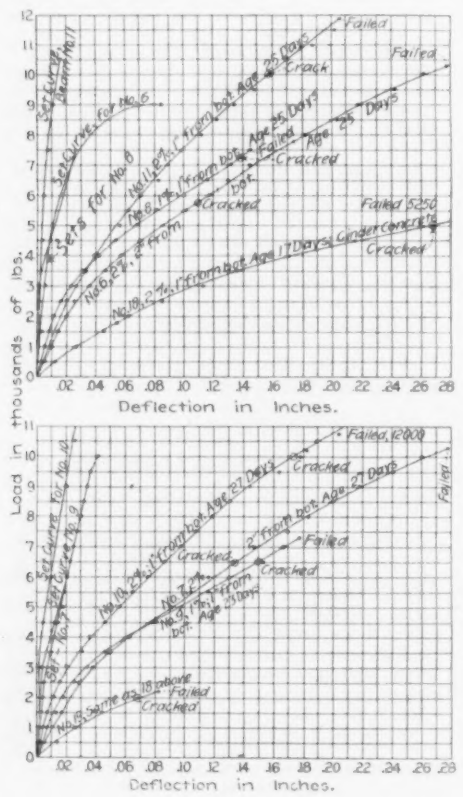

Load in thousands of lbs.
Deflection in Inches.
Failed
Cracked
Broke
Set Curve for No. 6
No. 6, 2 ins. from bot. Age 27 Days
No. 7, 2 ins. from bot. Age 27 Days
No. 8, 2 ins. from bot. Age 27 Days
No. 9, 2 ins. from bot. Age 27 Days
No. 10, 2 ins. from bot. Age 27 Days
No. 11, 2 ins. from bot. Age 27 Days
No. 12, 2 ins. from bot. Age 27 Days
No. 13, 2 ins. from bot. Age 27 Days
No. 14, 2 ins. from bot. Age 27 Days
No. 15, same as 13 in stone
No. 16, 2 ins. from bot. Age 27 Days
No. 17, 2 ins. from bot. Age 27 Days
No. 18, 2 ins. from bot. Age 27 Days
No. 19, 2 ins. from bot. Age 27 Days
No. 20, 2 ins. from bot. Age 27 Days
No. 21, 2 ins. from bot. Age 27 Days

Fig. 2.

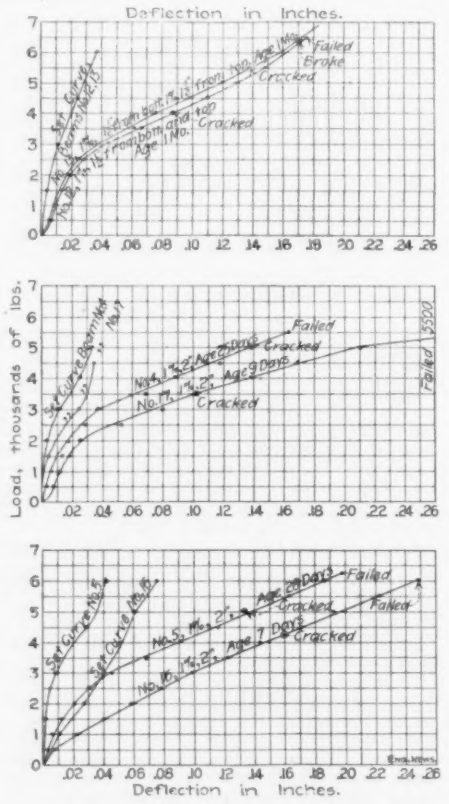

Deflection in Inches.
Load, thousands of lbs.
Set Curve
Failed
Cracked
Broke
No. 1, 2 ins. from bot. Age 9 Days
No. 2, 2 ins. from bot. Age 9 Days
No. 3, 2 ins. from bot. Age 7 Days
No. 4, 2 ins. from bot. Age 7 Days
No. 5, 2 ins. from bot. Age 7 Days
No. 6, 2 ins. from bot. Age 7 Days

Fig. 3.

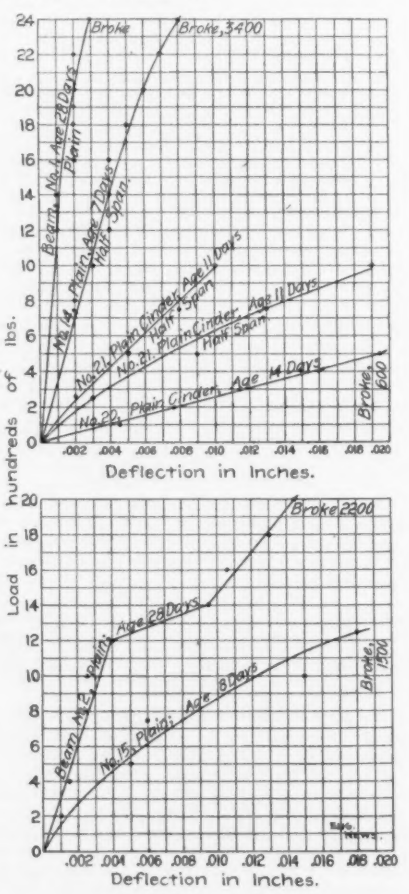

Load in hundreds of lbs.
Deflection in Inches.
Broke
Broke, 3400
Broke, 2200
Broke, 1900
Broke, 600
No. 14, Plain, Age 7 Days
No. 15, Plain, Age 28 Days
No. 16, Plain, Age 81 Days
No. 19, Plain Cinder, Age 11 Days
No. 20, Plain Cinder, Age 14 Days
No. 21, Plain Cinder, Age 11 Days
Gravel, No. 7, Plain, Age 28 Days
1 in. Span
Half Span

Fig. 4.

FIGS 2, 3 AND 4. LOAD-DEFLECTION DIAGRAMS OF REINFORCED CONCRETE BEAMS.

COMPRESSION AND TENSION TESTS.—Table II. gives the results of the compression and tension tests. The modulus of elasticity is computed with regard to the set experienced after previous loads. The stress quoted in case of reinforced concrete bars in tension is not corrected for the amount of the load carried by the reinforcement.

TABLE III.—Showing Adhesion of Round Steel Rods Inserted in Cubes of Stone Concrete.

Diameter of rod	7/16-in.	⅝-in.
Age	32 days.	35 days.
Depth of rod in concrete	6 ins.	6.4 ins.
No. of tests	3	3
Adhesion, lbs. per sq. in. of surface of rod:		
Maximum	735	780
Minimum	470	714
Average	636	756

TESTS OF ADHESION.—Table III. gives adhesion of round steel rods inserted in cubes of concrete. The values of adhesion represent the ultimate resistance in pounds per square inch of the surface of the rod nominally in contact with the concrete, against a direct tension. Sliding friction, after the adhesion was overcome, was from 60% to 70% of the adhesion. In many cases the concrete cubes were broken with a hammer after the test in order to determine the uniformity of the contact between the iron and the mortar. This contact obtained only in part, and was irregularly distributed over the surface of the rod.

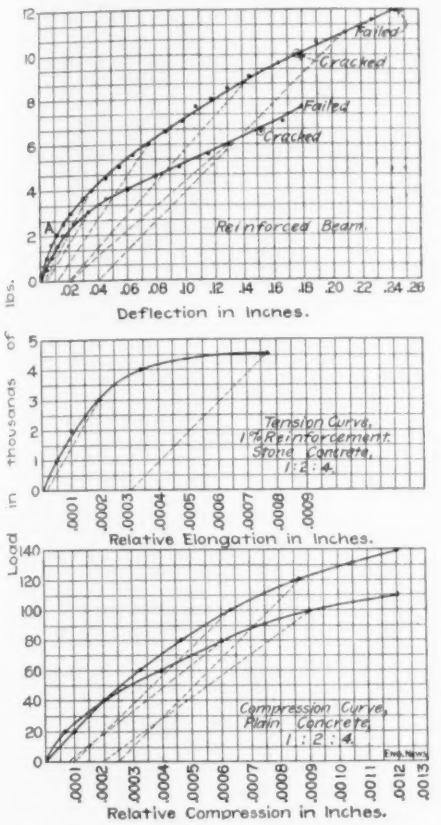


Fig. 5. Typical Diagrams for Reinforced Concrete Beams in Flexure.

TESTS OF IRON.—The metal used was common wrought-iron rods, 7-16-in. and ⅝-in. in diameter, respectively. The iron had a yield point of 36,000 lbs. per sq. in., and a modulus of elasticity of 29,000,000 lbs. per sq. in.

ANALYSIS OF RESULTS.

THE NATURE OF THE LOAD-DEFORMATION DIAGRAM OF REINFORCED CONCRETE BEAMS.—From an inspection of Figs. 2, 3 and 4 it will be seen that the diagram is nearly a straight line up to a load of from 1,500 to 3,000 lbs. in the various beams tested. At higher loads the deflection increases more rapidly. The diagram, however, again becomes nearly a straight line, the deflection then increasing uniformly with the load until at a load of from 4,000 to 10,000 lbs., a crack occurs in the concrete at the lower face of the beam. Beyond this point the deflection still increases uniformly with the load until the iron reinforcement reaches its elastic limit, at which time the deflection increases rapidly without any corresponding increase of load. These remarks refer mainly to the broken-stone beams one month old. Fig. 5 shows typical diagrams for concrete beams in flexure.

The three characteristic points of such a test are: the somewhat indefinite point A, where the curve first turns from the straight line; the point of first crack of the concrete, and the point at which the elastic limit of the reinforcing metal is reached. Table IV. is a summary of the facts of the test of the series of concrete beams listed with reference to these three points. The loads in Table IV. are in addition to the dead weight of the beams.

It may be said that in none of the stone-concrete beams from Nos. 1 to 17, inclusive, was there any indication that the compressive strength of the concrete was reached at the load at which the reinforcement failed. If steel reinforcement had been used the compressional strength of the concrete might have been developed. Nor, as might have been expected from the results in Table III., did the reinforcement pull out from the surrounding concrete. The cement, it may be noted, was strong enough to break the stones in the section of rupture of the beam.

RATIO OF THE MODULI OF ELASTICITY.—From Table II. it may be seen that the ratio of the modulus of elasticity of stone concrete in compression to that in tension was 2.17, corresponding to a stress of 750 lbs. per sq. in. in compression, and a stress of 300 lbs. per sq. in. in tension. It is to be noted that the ratio of the modulus of elasticity of the iron to that of the concrete in tension was 13.9. These values will apply to the beam when loaded to the point A. The important fact is the increased extensibility of the reinforced concrete in tension. Thus, while the plain concrete broke with an average extension of 1 : 7,000, the reinforced concrete broke with an average extension of 1 : 1,140. This fact has been determined by M. Considere, who says that plain concrete breaks in tension with an elongation of 1 part in 10,000, while reinforced concrete will withstand an extension of 1 part in 1,000 without rupture. The effect of reinforcement probably is to distribute the maximum elongation over the entire length of the bar, whereas, in case of plain concrete, the maximum elongation is confined to the fractured section.

TABLE IV.—Showing Results of Bending Tests of Reinforced Concrete Beams.

Beam No.	Kind.	Age in days.	Point A Load, lbs.	Point A Deff., in.	Crack Load, lbs.	Crack Deff., in.	Failure Load, lbs.	Failure Deff., in.
1	Plain.	28	1,200	.001	.....	....	2,400	.003
2	Plain.	28	1,200	.004	.....	....	2,200	.014
4	1%—2″	25	2,500	.026	5,000	.140	5,500	.163
5	1%—2″	28	2,000	.020	5,000	.136	6,250	.186
6	2%—2″	25	2,000	.020	5,750	.169	10,250	.278
7	2%—2″	27	2,500	.030	6,500	.133	10,250	.278
8	1%—1″	25	2,500	.016	7,250	.138	7,500	.145
9	1%—1″	23	2,500	.025	6,500	.150	7,300	.177
10	2%—1″	27	3,000	.022	10,000	.144	12,000	.247
11	2%—1″	25	3,000	.026	10,000	.158	11,800	.208
12	1%—1½″	30	2,000	.020	4,000	.087	6,500	.176
13	1%—1½″	30	2,000	.017	5,500	.138	6,400	.170
14*	Plain.	7	1,400	.004	.....	....	3,400	.008
15	Plain.	8	2,000	.0014	.....	....	1,500	.018
16	1%—2″	7	500	.007	4,250	.160	6,000	.248
17	1%—2″	9	1,400	.012	3,500	.161	5,500	.280
18†	2%—1″	17	1,000	.026	5,000	.268	5,250	.288
19†	2%—1″	17	500	.014	2,000	.068	2,300	.082
20†	Plain.	14	.....	....	.....	....	600	.023
21*	Plain.†	11	205	.003	.....	....	1,100	.019
21*	Plain.†	11	205	.002	.....	....	1,100	.016
22‡	3%—1″	6	4,000	.036	10,000	.124	11,500	.160

*½ span. †Cinder. ‡Gravel.

Accepting the tensile strength of the reinforced concrete as 300 lbs. per sq. in., and the elongation as 1 part in 1,000 at rupture, the equivalent modulus of elasticity of the reinforced concrete in tension is 300,000 lbs. per sq. in. at the stress of 300 lbs. per sq. in., i. e., at the point of cracking.

The new ratio of the modulus of elasticity of concrete in compression to that in tension is then 12.8 when these moduli are taken at stresses of 1,500 and 300 lbs. per sq. in. in compression and tension, respectively. The corresponding ratio for the iron and concrete is 96. These values will apply to the beams when loaded to the point of first crack. The bearing and importance of these ratios in the computation of the strength of such beams will be shown below.

Fig. 5 shows typical load-deformation curves also for concrete in compression and reinforced concrete in tension. From an examination of a number of stress-strain diagrams for concrete in compression, it appears that a parabolic arc closely approximates the actual curve.

ANALYSIS OF THE EFFECT OF VARIABLES.—In the beams tested, the variables were age, per cent. of reinforcement, position of reinforcement and material. In Fig. 6 the effect of these variables on the load and deflection corresponding to the first crack is graphically shown, and the following observations may be made: 1% of reinforcement 1-in. from the bottom of the beam increased the strength of a plain concrete beam from 2,200 to 7,400 lbs., and increased the flexibility from 0.01-in. center deflection to 0.14-in. center deflection. Increasing the reinforcement from 1% to 2% of the cross-sectional area increased the strength from 7,200 lbs. to 10,000 lbs., with only a slight increase in the flexibility. Placing the 1% reinforcement 2 ins. from the bottom face decreased the strength from 7,200 lbs. to 5,000 lbs., with a slight decrease in flexibility.

A cinder-concrete beam and a stone-concrete beam each reinforced with 2% of metal, 1-in. from the bottom face, had comparative strengths of 5,000 and 10,000 lbs., respectively, and a comparative flexibility of 0.26-in. and 0.16-in., respectively. In case of plain cinder and stone beams, the comparative strength was 600 lbs. and 1,800 lbs., and the comparatively flexibility was 0.023-in. and 0.016-in., respectively. It thus appears that reinforcing a beam with even 1% of steel gives it ten times its former flexibility and more than three times its former strength.

TABLE VI.—Showing Condition of Concrete Beams When Made and After Testing.

No.	Condition when made.	After test.	Wt. lbs.
1.	Concrete normal	Crack 4″ frm centr	480
2.	"	" at center....	485
4.	"	" 2″ frm centr	..
5.	"	" at center....	..
6.	"	" "	..
"	" rather wet.....	" "	..
8.	" liver consistency	" "	..
9.	" liver consistency	" "	..
10.	" very wet	" 6″ frm centr	..
11.	" rather dry	" 6″ frm centr	..
12.	" liver consistency	2 cracks, 2″ each side of center....	..
13.	" normal	Crack 3″ frm centr	..
14.	" rather dry	Tested at half span	..
15.	" normal	.....	480
16.	" rather dry	Crack 4″ frm centr	500
17.	" normal	" at center....	..
18.	" normal	" 3″ frm entr. Failed in compression later....	..
19.	" too dry	Crack 4″ frm centr	..
20.	" normal	" at center....	..
21.	" normal	Tested at half span	..
22.	Gravel concrete, 1:5.....	Failed suddenly by horizontal shear along reinforcement at end of beam	..

Note.—Nos. 4 to 17 are broken stone concrete, 1:2:4. Nos. 18 to 21 are cinder concrete, 1:2:4. No. 22, gravel concrete, 1:5.

THEORY.

Having established the facts developed from tests of the beams and their constituent materials, it remains to determine how far the theory, which will be advanced below, accords with the facts, and to recommend constants for use in the design of such beams. In the foregoing I has been the desire of the writer to so present the facts that the reader may make his own deductions should he disagree with those below.

This paper will not take up the development of the theory in much detail. The theory, the main analysis of which is indicated below, intends to account for the three characteristic points of the load-deflection curve shown in Fig. 5, viz., the point A, the load at first crack, and the load at failure.

The assumptions upon which the theory is based are mainly as follows: I. The cross sections of the beam

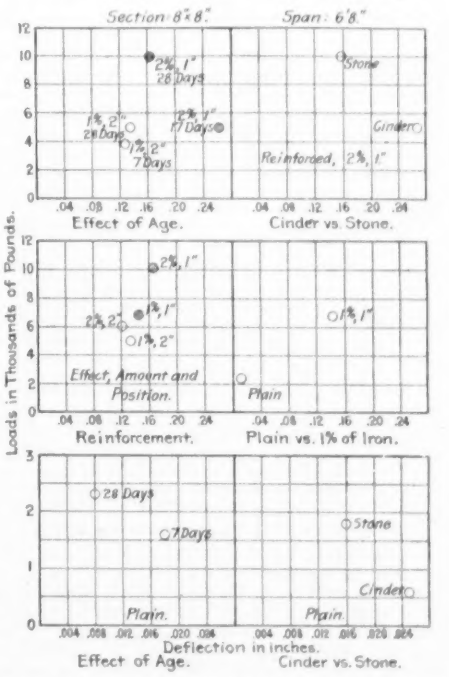


Fig. 6. Diagram Showing Effect of Age, Per Cent Reinforcement, Position of Reinforcement and Material on the Load and Deflection Corresponding to the First Crack in Concrete Beams.

TABLE V.—Showing Summary of Calculated and Observed Results for Reinforced Concrete Beams.

Kind of beam. Cement.	Sand.	Stone.	Reinforced Per ct. of.	Reinforced Position.	Load at Point A. Experiment. lbs.	Load at Point A. Theory lbs.	Load at First crack. Experiment. lbs.	Load at First crack. Theory lbs.	Failure. Experiment. lbs.	Failure. Theory lbs.	Stress; steel, lbs. pr sq.in. Point A.	Stress; steel, lbs. pr sq.in. 1st crack.	Concrete compress, lbs, per sq.in. Point A.	Concrete compress, lbs, per sq.in. 1st crack.	Concrete compress, lbs, per sq.in. Failure.	Value of x for position of neutral axis. Point A.	Value of x for position of neutral axis. 1st crack.	Value of x for position of neutral axis. Failure.
1	2	4	1%	2 ins.	2,000	2,103	5,000	4,925	5,500	5,470	2,000	17,320	457	1,556	1,639	.433	.302	.272
					2,500	.....	4,300	.....	6,200	.....	.....	.....	...	.....	.....	....	....	....
1	2	4	1%	1 in.	2,500	2,266	6,500	6,232	7,300	6,705	2,700	22,000	474	1,680	1,810	.441	.318	.298
					2,500	.....	7,200	.....	7,500	.....	.....	.....	...	.....	.....	....	....	....
1	2	4	2%	1 in.	3,000	2,648	10,000	9,680	12,000	12,515	2,760	21,570	516	2,188	2,710	.463	.378	.382
					3,000	.....	10,000	.....	11,800	.....	.....	.....	...	.....	.....	....	....	....
1	2	4	2%	2 ins.	2,000	2,400	5,750	7,123	10,250	10,840	1,950	16,550	408	1,937	3,050	.454	.354	.337
					2,500	.....	6,500	.....	10,250	.....	.....	.....	...	.....	.....	....	....	....

Note.—On account of dead weight of beam the theoretical gross loads should be reduced by one-half this dead weight in order to show true comparison as in Fig. 8.

remain plane surfaces. 2. The applied forces are perpendicular to the neutral surface of the beam. 3. The values of the moduli of elasticity obtained in simple tensional and compressional tests will apply to the material when under stress in the beams. (With reference to this connection between simple tests and flexural tests, it may be expected that the material in compression at the compressional face of the beam will bear a greater fiber stress than that indicated in a direct-compressional test; for these "outer fibers" of concrete are supported by material which has not reached its maximum strength.) 4. There is no slipping between the concrete and the steel reinforcement, and, 5, there are no initial stresses in the beam due to shrinkage or contraction of the concrete while setting. The analysis supposes fracture to be due to bending and not to horizontal shear.

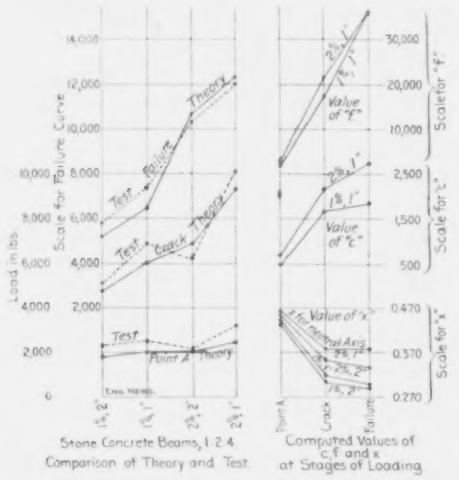


Fig. 8. Diagrams Showing Results of Comparison of Theory and Tests for Reinforced Concrete Beams.

If the cross sections are assumed to remain plane surfaces during flexure, the distortion of any fiber will be proportional to its distance from the neutral axis. It follows that the law of variation of stress will be represented by the stress-strain diagrams shown in Fig. 5.

Following the method of M. Considere in general by applying these diagrams to the cross section of a beam, we will have the representation of the stresses at different points of the cross section. These stresses in the concrete and that in the steel reinforcement are shown in Fig. 7.

For the sake of simplicity in the analysis, these stress-strain diagrams are assumed to be parabolic arcs, an assumption which is justified in the case of compressional stress-strain diagrams by an examination of a large number of tests recorded in "Report of Tests of Metals for the Year 1899." Referring to this figure, let

hx = the distance from the compression face to the neutral axis;
hu = the distance from the compression face to the center of gravity of the reinforcement;
p = the ratio of the area of steel to that of the cross-section of the beam;
E_s, E_c, E_t = the moduli of elasticity of the steel, concrete in compression and concrete in tension respectively;

$$n = \frac{E_c}{E};$$

$$m = \frac{E_s}{E_t};$$

f = stress in metal reinforcement;
c = compressive stress in outer fiber of concrete;
t = tensional stress in outer fiber of concrete.

Reinforcement is supposed to be in the tension flange alone.

E_c and E_t are measured at the stresses **c and t.**

The values of x, u, and p are ratios; p and u are at the control of the designer, while x depends on p, u, n, and m; n and m are fixed by the quality of the materials, and they change during flexure with the varying values of c, f, and t; that is, the modulus of elasticity of the concrete varies with the stress at which it is measured. For practical purposes of computation, however, the constant values of n and m may be used appropriate to the point A and to the point of cracking shown in Fig. 5.

On the assumption of plane cross sections during flexure, we may determine the ratio of f to c and f to t as follows:

$$c = \frac{t\,n\,x}{1 - x}. \qquad (1)$$

$$f = \frac{t\,m\,(u - x)}{1 - x}. \qquad (2)$$

Next, to locate the neutral axis; that is, to determine the value of x, we may equate the forces of tension and compression on the cross section, assuming, as before, that the stress-strain diagrams are arcs of parabolas. This:

$$\tfrac{2}{3}\,c\,x = \tfrac{2}{3}\,t\,(1 - x) + p\,f. \qquad (3)$$

inserting the values of c and f obtained above, we obtain the following quadratic:

$$\tfrac{2}{3}\,x^2\,n = \tfrac{2}{3}\,(1 - x)^2 + p\,m\,(u - x).$$

Solving the quadratic we have:

$$x = \frac{-\left\{\frac{4 + 3\,p\,m}{2}\right\} + \sqrt{4\,n + \frac{9}{4}\,p^2\,m^2 + p\,[6\,m\,(u\,[n - 1] + 1)]}}{2\,(n - 1)} \qquad (4)$$

Having obtained x we may compute c and f, and finally obtain the moment of resistance of the section. Taking moments about the neutral axis, we have: M =

$$t\,b\,h^2\left\{\frac{5}{12}\,(1 - x)^2 + \frac{5\,n\,x^2}{12\,(1 - x)} + p\,\frac{(u - x)^2}{1 - x}\,m\right\}. \qquad (5)$$

No useful development will result from the substitution in Eq. (5) of the value of x, obtained in general terms in Eq. 4. In practical computations n, m, u and p are given; x is then computed from (4); c and f computed from (1) and (2); finally the moment of resistance is computed from (5).

It may be said that these computations are somewhat tedious, but that it would be practicable for a designer after a few days work to tabulate all the beams falling in a given range of design. As in the case of problems in hydraulics, diagrams would greatly facilitate computation.

These equations given above are to be applied to compute the load at the point A. At the load corresponding to the cracking of the concrete in the tension face, these equations should be modified to correspond with the fact that the stress-strain diagram for the concrete in tension is more nearly a rectangle than a parabola. The difference, however, between the results at the time of the appearance of the crack due to the assumption of a rectangle or a parabola is small. With proper values of n and m the equations may be allowed to stand.

When, however, the crack having formed itself, extends throughout the lower region of the cross section, the equations must be modified by the omission of the effect of the tensional forces due to the resistance of the concrete under tension.

We have then:

$$\tfrac{2}{3}\,c\,x = p\,f \text{ or, } p\,\frac{E_s}{E_c}\,(u - x) = \tfrac{2}{3}\,x^2,$$

which serves to locate the neutral axis. When f is assumed to be the elastic limit of the reinforcing metal, c may be computed. As noted above, in the stone-concrete beams tested by the writer, the elastic limit of the iron was reached before the concrete failed in compression.

The resisting moment of the section is then

$$M = b\,h^2\left\{\frac{5}{12}\,c\,x^2 + p\,f\,(u - x)\right\}.$$

COMPARISON OF THEORY AND EXPERIMENT.

The loads carried by the stone-concrete beams as tested and recorded will now be compared with the loads computed under the application of the foregoing equations, using the elements of strength of the constituents as found by actual test. For this purpose the stone beams at the age of one month will be used. The values assumed are as follows: For the point A, n = 2, corresponding to c = stress below 750 lbs. per sq. in., and m = 12. The tensional strength of the concrete, t, is 300 lbs. per sq. in.

For the load at first crack, n = 12, corresponding to c = 1,500, m = 90; t is assumed to be 300, with an elongation of 1 : 1,000. For the point of failure, f is assumed to be 36,000. The results of this comparison of theory and experiment are given in Table V. and Fig. 8. The gross loads in Table V., from theory, are corrected to be net loads in Fig. 8 by subtraction of one-half of the weight of the beam. It may be said in general that the agreement between theory and experiment is more satisfactory than could be expected when the nature of the constituents is considered. It is a well-known fact that two pieces of concrete made under similar conditions do not show concordant tests. This satisfactory agreement in Table V. is partly due to the uniform results obtained from the tests on these beams; and also to the fact that the method of loading and support in flexure tests is mechanically more perfect than in case of tension and compression tests.

The agreement between tests and theory in case of point A could have been closer if the values of n and m had been slightly increased. The neutral axis rises as the loading increases, and f and c increase, as is shown in the right-hand side of Fig. 8.

CONCLUSIONS.

While realizing the imperfections in carrying out a somewhat ambitious program, which involved a great deal of labor and observation, and recognizing the fact that many minor points are left without discussion, the writer feels that he is justified in the following conclusions:

(1) The nature of the load-deflection diagram for the case of flexure of reinforced concrete beams is established (Fig. 5).

(2) The representation of the stresses over the cross section, as shown in Fig. 7, forms a working basis for analysis.

(3) The equations given for the purpose of computing the loads corresponding to the three characteristic points of the load-deflection diagram may be expected to give true results when the proper constants are inserted.

(4) In designing stone-concrete beams the load at point A may be computed using values of n and m of 2 and 12, respectively. The load at first crack in the tension face may be computed by using values of n and m of 12 and 90, respectively. The variation between theory and experiment was, on the average, nearly 8% when these constants were used.

(5) Using these equations and these values of n and m the designer may decide on the factor of safety obtaining in any given design.

(6) Indications are given of the effect of varying the materials and age on the strength and flexibility of such beams.

(7) The resistance due to tensional forces on the cross section under the working loads is an important element in the strength of a reinforced concrete beam.

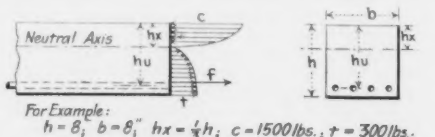


Fig. 7. Diagram Illustrating Nature of Stresses in a Concrete Beam.

THE PRESENT STATUS OF ARMOR AND ARMOR PLATES.*

The armored ship is more in evidence than ever. She will be protected at the water line by a broad belt starting well below water, where it meets the edge of the sloping armor deck and extending several feet above the water. The thickness of the belt may possibly be increased, but it seems probable that the increased resistance required to withstand the improvements in guns will take the form of thickening the slope of the armor deck.

Above the belt an armored box battery will occupy the midship part of the ship, and above this again, at the four corners of the battery, will be the principal secondary guns in turrets, the main armament being installed in double turrets or barbettes, as has now been for some time the case. The same arrangement seems likely to be followed for cruisers as well as for battleships.

With the adoption of turrets and box batteries the casemate will disappear. The thickening of the armor of the main barbettes is most necessary. The increase of the caliber of the secondary guns of battleships and first-class cruisers is imperative; much of the top hamper encumbering the upper decks of armored ships will have to go; also, possibly, the funnels will have to be lowered in action.

The year 1901 has not been notable as regards the manufacture of new types of armor. The Krupp process holds the field, and has been adopted by almost all nations, including England, America, Russia, Germany, Japan, Italy, in which latter country the Turni works have lately paid a compensation to Krupp, whose system has also been imported into France. It has certainly been declared in that country that the French type plates were found to be greatly superior to those made in France by the Krupp process; but the fact remains that the Russians, when buying plates in France, absolutely insisted on the Krupp process being used.

The Krupp process ("K. C.") is at its best for plates from 6 ins. to 9 ins. in thickness. It is much to be desired that the high resistance of the plates of medium thickness should, as experience is gained, be extended to thicker and thinner ones. But this has not been the case. The 11½-in. Carnegie plate, attacked by the 12-in. gun in competition with Gathman, was barely up to the average of plates of this thickness tried some years ago, and although some half-dozen trials of plates under 6 ins. have been recorded, none of them showed any specially high-resisting powers. Nor have there been many trials of very special interest apart from those in connection with the Gathman gun.

Except in the "Mikasa," the big guns are now comparatively unprotected against their fellows, whilst the secondary guns fare extremely well. The only exception to the latter is when the secondary guns are attacked by the 9.2-in. of the "King Edward" or the 8-in. of the "Georgia." Guns of this size will, therefore, be probably adopted in all new ships. The 7-in. and 7.5-in., with 7-in. and 8-in. penetration, respectively, will also supersede the 6-in. Moreover, the reintroduction of, say, 15-in. barbette plates seems more than likely. There is no justification for the present fashion of denuding barbettes to favor other parts of the ship. It will probably be found possible to make barbettes smaller, which would allow some thickening of the armor without increasing the weight.

*Extracted from "Brassey's Naval Annual" for 1902, by the "Journal of the United States Artillery" for May-June.